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LABORATORY STRESS-DEFORMATION CHARACTERISTICS OF SOILS UNDER STATIC LOADING

by Osman I. Ghazzaly and Raymond F. Dawson

Prepared by
UNIVERSITY OF TEXAS
Austin, Texas
for Langley Research Center

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ABSTRACT

A study of the laboratory stress-deformation characteristics of soils was performed. This study consists of two main parts. The first part includes a thorough literature survey of the subject. The factors believed to influence the stress-strain behavior of soils are presented. Some suggestions are made concerning the development of new test equipment and the modification of available equipment and procedures. Trials to measure lateral deformation of triaxial test specimens, using optical and photographic methods, are reported. Based on a literature review, evaluation of available test equipment is made. The modulus of deformation and lateral strain ratio are suggested as the proper parameters describing stress-strain properties of soils.

The second part of the investigation covers a laboratory study of stresses, strains, modulus of deformation, and lateral strain ratio for dry sands and saturated clays under various conditions. The variation of these quantities with the density of the sand, the confining pressure, the rates of strain usually encountered in laboratory testing of sand, the consolidation pressure and moisture content of clays, the stress history and degree of over-consolidation of the clay, plasticity index of the clay, and the level of stress and strain, is presented. The limited effect of small variations in the rate of strain on the angle of internal friction of a dry sand is reported. Also, a correlation is suggested among the moisture content, cohesion, and plasticity of normally-consolidated clays. Various forms of quick triaxial tests, under one-cycle static load conditions, were used. A method is given

to eliminate the effect of friction of the loading rod in triaxial tests. A procedure for measuring lateral deformations of vacuum triaxial test specimens of dry sand, and unconfined clay samples, using extensometers is also outlined. All test specimens were fabricated in the laboratory.

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SYMBOLS AND NOTATIONS

A	=	cross-sectional area of the test specimen
A_o	=	original cross-sectional area of the test specimen
c	=	cohesion
cu.in.	=	cubic inch
D	=	diameter of the test specimen
D_o	=	original diameter of the test speciman
E	=	modulus of deformation
E'	=	modulus of elasticity
E_{25}	=	modulus of deformation at one quarter the maximum deviator stress
E_{50}	=	modulus of deformation at half the maximum deviator stress
E_{75}	=	modulus of deformation at three quarters the maximum deviator stress
E_{100}	=	modulus of deformation at the maximum deviator stress
E_i	=	initial tangent modulus of deformation
F.S.	=	factor of safety
G	=	shear modulus of deformation
G'	=	modulus of rigidity
H	=	height of the test specimen
H_o	=	original height of the test speciman
in.	=	inch
k_o	=	coefficient of earth pressure at rest
L.L.	=	liquid limit
lb	=	pound

min = minute

N.C.C. = normally-consolidated clay

No. = number

O.C.C. = over-consolidated clay

O.C.R. = over-consolidation ratio

P = axial load acting on the specimen, in excess of the confining pressure

P.I. = plasticity index

pcf = pounds per cubic foot

psi = pounds per square inch

R.S. = rate of strain

sq.in. = square inch

T.M. = Taylor Marl

V = volume of the test speciman

V_o = original volume of the test speciman

V.S.C. = Vicksburg silty clay

w = moisture content

γ = density

Δ = axial deformation

Δ_1 = lateral deformation

ΔV = change in volume of the test specimen

ϵ = axial strain

ϵ_{50} = axial strain at half the maximum deviator stress

ϵ_{75} = axial strain at three quarters the maximum deviator stress

ϵ_{100} = axial strain at the maximum deviator stress

ϵ_i = axial strain at which the stress-strain curve is no longer a straight line

ϵ_1	=	lateral strain
μ	=	lateral strain ratio
μ'	=	Poisson's ratio
μ_{50}	=	lateral strain ratio at half the maximum deviator stress
μ_{100}	=	lateral strain ratio at the maximum deviator stress
σ	=	axial, or normal, stress
σ_1	=	major principal stress
σ_2	=	intermediate principal stress
σ_3	=	minor principal stress
σ_c	=	consolidation pressure
σ_o	=	over-consolidation pressure
σ_Δ	=	deviator stress
$\sigma_{\Delta_{0.1}}$	=	deviator stress at 0.1 in. per in. axial strain
$\sigma_{\Delta_{0.2}}$	=	deviator stress at 0.2 in. per in. axial strain
$\sigma_{\Delta_{50}}$	=	half the maximum deviator stress
$\sigma_{\Delta_{75}}$	=	three quarters the maximum deviator stress
σ_{Δ_i}	=	deviator stress at which the stress-strain curve ceases to be a straight line
$\sigma_{\Delta_{max}}$	=	maximum deviator stress
τ	=	shear stress
ϕ	=	angle of internal friction
/	=	per
%	=	per cent

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CHAPTER I

INTRODUCTION

1.1 The Significance of Stress-Deformation Characteristics of Soils

The determination and study of stress-deformation characteristics of soils are of basic importance in many areas of soil mechanics and foundation engineering. Stress distribution and soil deformation problems as well as soil-structure interaction problems involve the stress-strain behavior of soils. Earth pressure and stability problems, and the design of foundations for various purposes, whether subject to static or transient loading, require the knowledge of such behavior. Settlement of footings, design of foundations for power plants, dams, or missile-launching silos, and airfield or highway pavement design are but few examples of the many applications of such studies.

The invention of electronic digital computers in recent years, and their widespread use to solve soil engineering problems involving stress-deformation behavior of soils, made the study of such behavior even more worthwhile. It is now possible to solve a variety of complicated problems in a few seconds that only a few years ago we would not have attempted to solve because of the tremendous amount of computations involved requiring great effort and time.

Theoretical investigations are based, in general, on the behavior of a perfectly elastic soil with a linear stress-strain curve. This is not absolutely correct, however, and an investigation of the stress-strain properties of different soils is definitely required.

Some investigators believe that the theory of plasticity may be used to solve stability problems dealing with the ultimate failure of a mass. The deformations of a soil mass are determined by the theory of elasticity (21)*. The question of how far the theory of elasticity can be applied to soils has yet to be fully answered but the fact still remains that lack of elastic constants and the uncertain delineation of the elastic and plastic states of a soil enormously hinder the progress in such application. Other investigators advocate the consideration of soil as a construction material, comparable to concrete, for example (43). Unfortunately there is not sufficient stress-strain data for soils that can lead to a rational design of earth structures.

1.2 Laboratory and Field Stress-Strain Behavior of Soils

At the present time there is no test in soil mechanics that exactly reproduces all natural conditions. This leads to laboratory stress-strain curves which are somewhat different than the actual ones for the same soil mass in the field. The problem of estimating soil properties under natural conditions from laboratory test data is one that probably will always be among the most complex in soil engineering and its solution will always be subject to many pitfalls. Arriving at a general stress-strain theory for soils that involves a large, unknown number of complex stress-strain-time relationships must be acknowledged as an impossible goal (37).

Prediction of field behavior from laboratory data can be accomplished when new laboratory and field testing equipment and techniques, that fully reproduce field conditions, are developed. Trials along this line should be encouraged. Until this goal is reached, however, modification of present day

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*See bibliography, pp 172-175.

equipment must be continued to eliminate much of its shortcomings. The author firmly believes that the determination of the actual behavior of soils in the field may be accomplished by correlations between laboratory stress-strain properties and well documented case records and results of full-scale field tests on various foundation elements. Thus a comprehensive study of the subject at the present time would include, among other things, the use of available equipment to investigate fully the laboratory stress-strain characteristics of soils. Skempton (33) carried out a similar study on the London clay using the unconfined compression test.

1.3 Objectives and Scope of the Investigation

The urgent need for an intensive investigation of the stress-deformation characteristics of soils has been pointed out in the preceeding articles. The present study is intended to be a step forward along this line. It consists of two main parts. The first part includes a literature review which summarizes our present state of knowledge in this area of soil mechanics. The factors believed to influence the stress-strain behavior of soils are presented. Some suggestions are made concerning the development of new test equipment and the modification of available equipment and procedures. Trials to measure lateral deformations of triaxial test specimens, using optical and photographic methods, are reported. Based on the literature survey, an evaluation of available test equipment is made. The modulus of deformation and the lateral strain ratio are suggested as the two parameters necessary to describe the stress-strain properties of a soil. The second part of this investigation covers a laboratory study of stress-strain properties of some sands and clays under one-cycle static load conditions using various forms of the quick triaxial test. A method for eliminating the effect of friction of the loading rod in the triaxial test is presented. Also a procedure is given for measuring lateral deformation of vacuum triaxial, and unconfined, test specimens using dial extensometers.

It was noticed in almost all similar studies reported in the literature, that no attention was given to the deformation of the soil and that the strength was the main interest. It was therefore decided to study both stresses and strains during the entire range of the test for the various soils. It is hoped that the accumulation of such information from the present and future studies will enable the selection of proper stress-strain curves of soils, if no sample is available. The information gained helps also in understanding the stress-deformation behavior of soils under various conditions. In this respect the present laboratory investigation is believed to outline some of the necessary steps to reach that goal.

The factors that were studied in the tests on dry sands are the density of the sand, the confining pressure, and the rates of strain usually encountered in laboratory testing. For clays, both normally-consolidated and over-consolidated samples were tested. The effect of moisture content, consolidation pressure, confining pressure, plasticity, and degree of over-consolidation are presented.

The modulus of deformation and lateral strain ratio for some soils were also studied. Their variation with the different factors mentioned in the previous paragraph, as well as with the level of stress and strain, is reported.

Attention was also given to the soil strength. For normally consolidated clays, a relation is suggested among the cohesion, moisture content, and plasticity. The effect of small variations in the rate of strain on the angle of internal friction of a dry sand is also presented.

PART A

GENERAL CONSIDERATIONS

CHAPTER II

THEORY OF ELASTICITY AND ITS APPLICATION TO SOILS

2.1 Considerations Based on the Theory of Elasticity

Most theoretical investigators assume that soils are elastic, homogeneous and isotropic. The relations between the states of stress and strain at a point of an elastic body are expressed by Hooke's law which states that stress is proportional to strain. The fact that the deformation produced in a body upon application of load is proportional to it, independent of time, and recoverable after the removal of this load is true in most elastic materials only for small strains. The definition of elasticity does not require the stress-strain curve of a material to be linear. This is the assumption made, however, in the linearized theory of elasticity which assumes small deformations (4). Hooke's law in this case assumes the material to be elastic, isotropic, and homogemeous. The generalized form of Hooke's law (48), which gives the linear relations between stress and strain components, can be given by the following six formulas:

$$
\begin{aligned}
\sigma_{xx} &= C_{11}\,\epsilon_{xx} + C_{12}\,\epsilon_{yy} + C_{13}\,\epsilon_{zz} + C_{14}\,\gamma_{xy} + C_{15}\,\gamma_{xz} + C_{16}\,\gamma_{yz} \\
\sigma_{yy} &= C_{21}\,\epsilon_{xx} + C_{22}\,\epsilon_{yy} + C_{23}\,\epsilon_{zz} + C_{24}\,\gamma_{xy} + C_{25}\,\gamma_{xz} + C_{26}\,\gamma_{yz} \\
\sigma_{zz} &= C_{31}\,\epsilon_{xx} + C_{32}\,\epsilon_{yy} + C_{33}\,\epsilon_{zz} + C_{34}\,\gamma_{xy} + C_{35}\,\gamma_{xz} + C_{36}\,\gamma_{yz} \\
\tau_{yz} &= C_{41}\,\epsilon_{xx} + C_{42}\,\epsilon_{yy} + C_{43}\,\epsilon_{zz} + C_{44}\,\gamma_{xy} + C_{45}\,\gamma_{xz} + C_{46}\,\gamma_{yz} \\
\tau_{xz} &= C_{51}\,\epsilon_{xx} + C_{52}\,\epsilon_{yy} + C_{53}\,\epsilon_{zz} + C_{54}\,\gamma_{xy} + C_{55}\,\gamma_{xz} + C_{56}\,\gamma_{yz} \\
\tau_{xy} &= C_{61}\,\epsilon_{xx} + C_{62}\,\epsilon_{yy} + C_{63}\,\epsilon_{zz} + C_{64}\,\gamma_{xy} + C_{65}\,\gamma_{xz} + C_{66}\,\gamma_{yz}
\end{aligned} \qquad (1)
$$

where

C denotes elastic constants.

ϵ_{xx}, ϵ_{yy}, ϵ_{zz} = normal strains.

σ_{xx}, σ_{yy}, σ_{zz} = normal stresses.

γ_{yz}, γ_{xz}, γ_{xy} = shear strains.

τ_{yz}, τ_{xz}, τ_{xy} = shear stresses.

The normal and shear stresses acting on the element shown in Fig. 1 are those used in the generalized Hooke's law. Equilibrium requires that $\tau_{yx} = \tau_{xy}$, $\tau_{yz} = \tau_{zy}$, and $\tau_{xz} = \tau_{zx}$. Thus the state of stress can be completely specified by only three shear and three normal components of stress. For each stress component there is a corresponding strain component. In anisotropic materials a pure stress σ_{xx} does not necessarily produce a pure strain ϵ_{xx}, but it may cause any other type of strain (48).

This generalized form of Hooke's law gives thirty-six possible elastic constants. However, for an isotropic material the number is reduced to twenty-one independent elastic constants which are necessary in the case of an isotropic body without symmetry (where $C_{12} = C_{21}$, $C_{13} = C_{23}$, and so on). The number of elastic constants will decrease with increasing symmetry and in the case of cubic symmetry only three elastic constants are required to define the states of stress and strain. These three are (48):

E' : the modulus of elasticity defined as the ratio of stress to strain

μ' : Poisson's ratio defined as the ratio of the strain in the direction perpendicular to the applied load to that in the direction of load.

G' : the shear modulus of elasticity defined as the ratio of shear stress to shear strain. It may also be termed the modulus of rigidity.

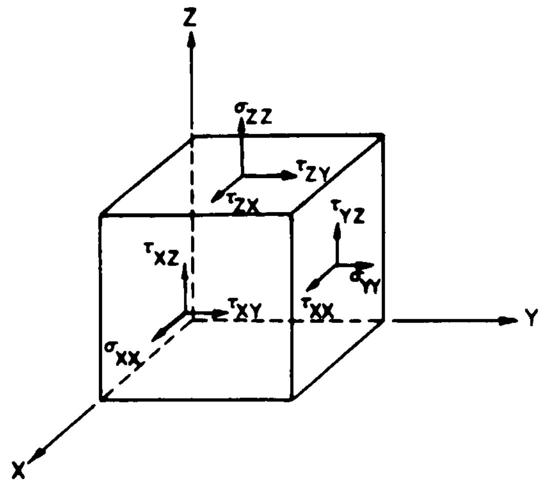

FIG. 1. NORMAL AND SHEAR STRESSES ACTING ON AN ELEMENT

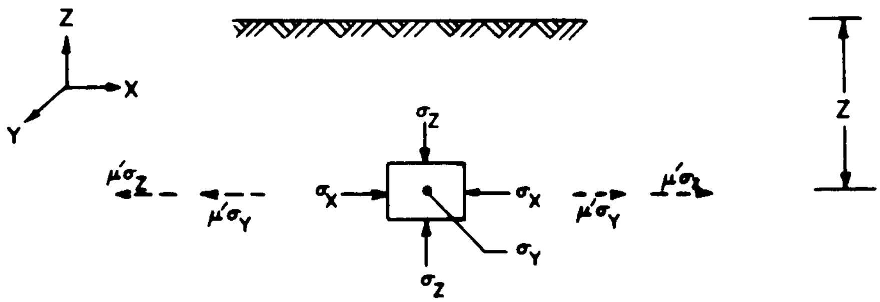

FIG.2. AN ELEMENT CONFINED IN THE X AND Y DIRECTIONS

Although there are three fundamental constants, in the above case, only two of these are independent because of the following relation among them:

$$G' = \frac{E'}{2(1+\mu')} \qquad (2)$$

Thus in the case of elastic, homogeneous, isotropic bodies, and with small deformations, that is in the case of applying the assumptions of the linearized Hooke's law, the system of stresses and strains within a mass is completely defined with only two independent elastic constants. Generally the modulus of elasticity E' and Poisson's ratio μ' are used. Of course, the boundary conditions in any specific problem together with compatibility conditions; that is, the requirement that deformations of a body be finite, continuous, and single valued, have to be satisfied.

When the deformations increase beyond a certain limit, the application of the linearized theory of elasticity becomes no longer possible. For larger deformations the linear relationship between stress and strain no longer exists. In this range most engineering materials show both elastic and plastic deformations. Elastic deformations are recoverable upon removal of the load that caused them while plastic deformations are not; that is, they are permanent. Elastic recovery is due to the action of interatomic and intermolecular forces while plastic deformation is the result of permanent displacement of atoms or molecules (48). For most materials the elastic deformation precedes plastic strain.

There are basically two types of deformations (48):

1. Pure elongation or contraction.
2. Pure shear action, or sliding.

Any deformation can be given as a combination of these two. Both stress and strain can be shown graphically by the Mohr circle.

The volumetric strain (40) occurring in an element of an elastic mass subjected to axial compression is given by the formula:

$$\frac{\Delta v}{v} = \frac{1-2\mu'}{E'} \, (\sigma_1 + \sigma_2 + \sigma_3) \quad \ldots \ldots \ldots \ldots \ldots \ldots \quad (3)$$

where

Δv = volume change of the element.

v = original volume of the element.

σ = axial compressive stress.

E' = modulus of elasticity.

μ' = Poisson's ratio.

Poisson's ratio equals zero in the case of complete lateral confinement; that is, when there is no deformation taking place in a direction perpendicular to the applied load. It is equal to one-half when no volume changes occur during compression. From equation number (3) it can be seen that μ' is less than one-half in case of volume decrease and greater than one-half when there is volume increase.

2.2 Limitations and Possibilities in the Application of the Theory of Elasticity to Soils.

There is doubt as to whether soils can truly be termed elastic and whether the theory of elasticity can be used in the field of soil mechanics. Further, there are factors required in such application for which there is insufficient background. The essential elastic constants of soils; namely, the modulus of elasticity and Poisson's ratio, are treated by assumptions largely unsubstantiated by factual knowledge. Two and three dimensional solutions using the theory of elasticity have found many applications since

Hooke's law was developed. Bousinnesq, Westergard, and Newmark, among others, have applied this theory to problems in the field of soil mechanics and determined the stress distribution due to loads on a semi-infinite elastic mass (43).

Elastic response to stress is the basis for applying the theory of elasticity for any material and is also a requirement in the determination of elastic constants. This completely elastic state is seldom developed in soils which have a dual elastic-plastic nature that must be examined to determine the validity of the elastic approach used extensively in soil mechanics. In problems dealing with soils, both elastic and plastic properties of soils must be treated. Failure to acknowledge the importance of both elasticity and plasticity in soils is the major impediment to the rational application of the theory of elasticity to soils.

Soils in general follow complicated stress-strain-time laws and this together with the fact that soils are rarely homogeneous and isotropic makes it difficult to predict stresses and displacements accurately. Thus to accomplish this task, it is necessary to accept over-simplified models of soil behavior in order to arrive at an engineering approximation. The accepted model of soil is the homogeneous, isotropic, and elastic half space. The results of the Waterways Experiment Station tests suggest that, in clay soils at least, computation of stresses by the theory of elasticity is admissible (1). If a clay is compacted, the remolding destroys any structural anisotropy and results in a material which is generally isotropic (1).

Expressing the mechanical properties of a soil by coefficients which can be used in theoretical computations is difficult because these coefficients are not constants but vary under different conditions. There are two main problems in soil mechanics, stress-strain relationship of a soil in an elastic state and the shear strength in a plastic state (5). It is customary to assume as constants for the elastic case the modulus of elasticity and Poisson's ratio, and for the plastic state the angle of internal friction ϕ and cohesion c (5).

Terzaghi (40) states that if the factor of safety of a mass of soil, with respect to failure by plastic flow, exceeds a value of about three the state of stress in the soil is likely to be more or less similar to the state of stress computed on the assumption that the soil is perfectly elastic. Hence the state of stress in a mass of soil under the influence of moderate stresses can be estimated by means of the theory of elasticity. The importance of the error associated with the results of the computation depends chiefly on the extent to which the real stress-strain relations depart from Hooke's law. This departure increases rapidly as the state of plastic equilibrium is approached.

Compression of the natural soil is caused mainly by a decrease in the porosity and not a reduction of the grain size (5). For this reason soil behaves quite differently under loading and unloading.

Soils exhibit stress-strain relations which are often curved throughout their entire length. The variable and seemingly unpredictable stress-strain properties of soils preclude the selection of proper elastic moduli (11). Thus the actual deformation characteristics of soils are required.

The conditions existing in any problem are seldom exactly comparable to the conditions upon which elastic formulas are based. If soil specimens can be subjected to the stresses that will be applied to the soil mass at a proposed project, and a straight line plot is obtained when the observed strains are drawn versus stresses, then the theory of elasticity can be applied (37). Effects of any factor involving the time element bring in complications which represent limitations to the validity of the application of the theory of elasticity to soils.

In the application of the theory of elasticity to soils no consideration is given to the general non-linear character of soils, nor are the effects of anisotropy or non-homogeneous conditions taken into account. A study of these effects represents an important area of research which would have immediate practical applications.

The assumptions of continuity and isotropy in the soil particle skeleton, or structure, are far reaching but inescapable. If any theory of discontinuous behavior in the interior of the soil body were seriously proposed, it would be necessary to obtain radically different data (29). At present there is scarcely sufficient reliable data to determine isotropic constants for soils and quantitative analysis of anisotropy must be left until the nature of any deviation from isotropic predictions is clearly established.

In the application of the theory of elasticity, the assumption that the material is homogeneous and isotropic can be justified to some extent in the case of soils, although it is not strictly correct in this respect (43).

Spillers and Stoll (35) suggest that in order to formulate a continuum model for a soil mass, which from a theoretical point of view is an inelastic continuum, a simple model must be used to start with (the simplest continuum model is a homogeneous, isotropic and elastic solid analyzed for conditions of small strain) and desired properties added until a model is achieved which is sufficiently detailed to represent the real material (plasticity can be added to the soil model as a first modifying effect). It is necessary to decide which properties are to be included and this can only be determined from a study of the stress-deformation and other required characteristics for various soils.

The accuracy of predicting settlements by elastic methods is influenced by time effects, anisotropy, creep, and inelasticity of soils (36).

In all soils, deformations on first loading are largely irreversible (28). But in a cyclic loading test on clay, if the load were increased to a certain value, a great part of the permanent deformation would occur. The unloading and reloading to a value within the prestress, therefore, will be mostly elastic. The true elastic modulus will be that calculated from the reloading curve, which may be either different from or identical to the modulus for unloading (24). The modulus defined by the virgin loading involves both elastic and plastic effects and is not a true modulus of elasticity but should be considered in some practical problems. Cyclic loading tests on clay indicated that the recompression curves of stress-strain diagrams are straight up to certain limits of stress below failure and the modulus of recompression appears to be the same for each cycle in the same test (24). The unloading curves, however, are essentially non-linear.

Werner (43) reports similar observation and states that if a soil is subjected to stress, deformation is evidenced in each of the three principal planes. If the stress is removed, a certain amount of deformation is recovered (elastic). As the stress is applied and removed a number of times, the amount of plastic deformation decreases until a point is reached where each cycle essentially tracks the values of the previous one, and the slope of the stress-strain curve remains constant. At this point the soil is said to be elastic within the range of its stress experience. As the stress situation is extended to greater values, plasticity must again be removed by repetitive loading. An elastic state would have to be reached before any elastic constants can be determined to be used in an intelligent application of the theory of elasticity.

CHAPTER III

DETERMINATION OF STRESS-STRAIN CHARACTERISTICS OF SOILS

3.1 General

The problem of determining the stresses and strains within a soil mass subjected to some form of loading is one of the most important in soil mechanics. There are many theories developed to express the state of stress and defo rmation at any point within a soil mass. Most of these are based on the theory of elasticity, for example, the work done by Buossinesq, Westergard, or Newmark. Some researchers tried to take into account the effect of soil plasticity or other soils properties, but all of them used simplifying assumptions to help reduce the great number of variables involved. Some of these attempts are given in references (1), (5), (11), (35) and (36), which are only given as examples of such attempts.

In all cases adequate knowledge of the stress-strain properties of the soil mass is vital before any rational analysis of stresses and deformation within the loaded mass can be accomplished. This emphasizes the need for a laboratory study of the axial as well as lateral stress-deformation behavior of soils under various conditions. The author believes that two quantities analogeous to the modulus of elasticity E' and Poisson's ratio μ' in the case of elastic materials, will represent the required soil properties. These two quanitities, when determined for various soils, are not expected to be constant in all cases. Thus a study of their pattern of variation, under various conditions, is required. Experimental studies in this area of soil mechanics are few and incomplete in covering the wide scope of the subject and any investigation along this line would be helpful in exploring this complex problem and in adding to our present insufficient knowledge about the subject.

3.2 Factors Influencing Stress-Strain Properties of Soils

There are many factors that affect the stress-deformation behavior of soils. A complete and comprehensive study of the influence of these factors is very important for the determination of such behavior under various conditions. The following variables are believed to be among these factors:

1. Type of soil; for example, clay, silt, or sand.
2. Moisture content of the soil, which is most important for clays.
3. Confining pressure, or depth effect.
4. Density of the soil (especially in the case of sands).
5. Plasticity and activity of the soil (for clays).
6. Grain size distribution, shape, and texture of the soil particles (this factor is important for sands.)
7. Degree of saturation of the soil with water.
8. Degree of remolding or disturbance to which the soil structure is subjected, which is important for clays. The thixotropy of the soil can definitely be entered into the picture here.
9. Level of stress, or the level of strain.
10. Type of loading, whether static, cyclic, repeated or dynamic loading. In each case there are several things to consider, for example, in the case of repeated loading, the magnitude, frequency, and duration of the applied load are among the factors to be investigated.
11. Type of test. There are many tests used in the laboratory determination of stress-strain properties of soils. For example, there are

various types of triaxial compression tests (undrained, consolidated undrained, and drained tests). There is also the unconfined compression test and others. The size of the specimen used in any test may also have some effect on the results.

12. Time. Possible effects of time can be separated into the following categories:

    (A) Thixotropic effects (for example, time of storage before testing).

    (B) Aging effects (length of time allowed for consolidation, other than primary, prior to loading).

    (C) Strain-rate effects, that is, the speed with which the load is applied.

    (D) Creep effects (or plastic flow under constant stress).

13. Stress history of the soil. For the case of clays consideration has to be given to whether the soil is normally consolidated or over-consolidated and to the degree of over-consolidation (as measured by the over-consolidation ratio, defined as the ratio of the over-consolidation pressure to that acting on the soil at present.)

14. Types of stress system (22). Both the magnitude and direction of principal stresses should be considered. The three basic types of stress systems that can be applied during shear, which depend on the relative magnitude of the applied intermediate principal stress, σ_2 , are:

    (A) Triaxial compression, in which the intermediate principal stress σ_2 equals the minor principal stress σ_3.

    (B) Triaxial extension, in which $\sigma_2 = \sigma_1$, the major principal stress.

(C) Plane strain, in which σ_2 is intermediate between σ_1 and σ_3, and in which all strains in the soil are parallel to the plane of σ_1 and σ_3.

15. Type of consolidation, whether isotropic, that is, equal all-round consolidation pressure, or anisotropic, which could either be accomplished by allowing lateral deformation to occur or with no lateral deformation taking place during consolidation in a case of complete confinement.

16. The pore pressures developed and volume changes occurring in a soil during loading. These two considerations are actually tied closely to most of the above factors. The strength of the soil has an effect and is related to the preceeding variables.

3.3 Modulus of Deformation

The modulus of deformation of a soil E is defined as the ratio of the deviator stress to axial strain at any point on the stress-strain curve for that soil. Since a big portion of any stress-strain curve for a soil is generally non-linear, then the modulus of deformation is not a constant. This is the case for virgin stress-strain curves. However, if the load is repeated, it may be possible to eliminate most of the soil plasticity and arrive at a single-valued modulus of deformation. The modulus in this case may be termed the hysteresis modulus (11). The effect of repeated loading on the stress-strain properties of some soils was mentioned in Art. 2.2, but more research is definitely required in this area to draw final conclusions.

For a laboratory one-cycle stress-strain curve one of the following quantities may be used to represent the modulus of deformation (11, 46) of the soil:

1. The initial tangent modulus: determined from the slope of initial straight line portion of the stress-strain curve, or the slope of the initial tangent to the curve.
2. The secant modulus: taken at any point on the stress-strain curve. The most common value is E_{50} taken at half the ultimate deviator stress.

According to the type of loading used in determining the modulus of deformation for any soil, it can be classified as (i) dynamic modulus of deformation, (ii) hysteresis modulus (for repeated loading), or (iii) static modulus of deformation. The dynamic modulus of deformation can be obtained from a dynamic stress-strain curve determined in the laboratory. It can also be determined by using data on the velocity of propagation of seismic waves and elastic theories for computation. For repeated loading the hysteresis modulus (sometimes known as the modulus of elasticity) is determined at the end of a repetitive loading period during which small on-off load increments are used until plastic deformation is eliminated (45).

Various tests and equipment are used to determine the modulus of deformation. The triaxial test is one example of laboratory testing. There are many forms of triaxial tests - for example, triaxial compression, triaxial extention, and vacuum triaxial tests. The drainage conditions during the test can be varied to allow, or prevent, consolidation. In the conventional triaxial compression test the confining pressure is kept constant during the test. Another form of the triaxial test is suggested (20, 46) where σ_3 is varied so that the ratio σ_3/σ_1 is always the same. This type of test may be used to get anisotropic consolidation of the sample and is believed to be a better representation of field conditions in some instances. The modulus of deformation may vary when determined by different tests. In the field of soil mechanics no test procedure has yet been developed that measures directly the desired soil parameters. Thus the required properties must be determined from the one test that better represents field conditions to get the most accurate results. Other tests that can be used to determine the modulus of deformation, beside the various forms of the triaxial test given above, include:

1. A test to determine the constrained modulus of deformation in which the soil is placed in a container to prevent lateral deformation (the consolidometer ring is an example). This is a one-dimensional compression case with complete confinement that is believed to represent soil conditions in deep strata where lateral deformation is negligible. The friction along the sides of the container and between the soil and end plates should be minimized to assure uniform stresses and strains throughout the sample. In this test no failure of the sample is possible under the prevailing conditions. Dynamic tests of this type have been performed by Wilson and Sibley (46) and by Heierli (17). Other types of loading are also believed possible in this test. Also the measurement of the lateral load transmitted to the sides of the container seems feasible.
2. The unconfined modulus of deformation can be determined for clays only using the unconfined compression test. It is recommended to be used only for empirical correlations.
3. The modulus of deformation for the case of plane strain: determined from a plane strain apparatus such as the one described by Cornforth (8) where the soil is loaded under conditions of plane strain. Such a condition is often encountered in practical soil mechanics problems.

The plane strain apparatus mentioned above needs much improvement and is still under development.

The results obtained by various investigators point out the fact that much needs to be done as far as the modulus of deformation of the soil is concerned.

The shear modulus of deformation G can be determined from a test in which the soil specimen is subjected to a shear stress and the developed shear deformations measured subsequently. A test in which a torque is applied to a solid or hollow cylindrical soil specimen (6, 15) may be used.

3.4 Lateral Strain Ratio

The lateral strain ratio μ is defined as the ratio of the strain in a direction perpendicular to the applied load (lateral strain in a triaxial specimen) to that in a direction parallel to load (axial strain). It is analogous to Poisson's ratio for elastic materials, but is not believed to be a constant for soils under various conditions.

Lateral strain ratios for soils may be determined in the laboratory by either direct or indirect methods. Direct methods involve the measurement of axial and lateral deformations of a test specimen. The apparatus that can be used is triaxial, unconfined, plane strain, or some other test equipment suited for the purpose. Indirect methods involve the determination of certain quantities, then using some relations that tie them with the lateral strain ratio, and which are based on simplifying assumptions (as will be shown later on). In any case the determination of lateral strain ratios for soils is a complicated task and in some instances requires reliance upon various assumptions and is quite tedious and time consuming with present-day facilities.

The measurement of lateral deformations in a triaxial test is very difficult and may be accomplished in one of the following ways:

1. Optical methods of measuring the cross-section of a sample are suggested by Escario and Uriel (12), who state that errors can be minimized when two diametrically opposed readings are made. The author tried measuring the lateral deformation of a sand specimen in a triaxial test, at various strain levels during the test, using a theodolite placed at a known distance from the specimen. After several trials it was concluded that there were difficulties involved that made such measurement impractical. The speed of the test prevented accurate measurement, as the person taking the readings could not keep up with the normal rate of strain used (1% per minute). Another factor that should be considered in these measurements is the fact that the sample during the test generally does not remain a perfect cylinder. At least two theodolites, perpendicular to each other, are recommended to get the measurements at any one level of the specimen. In any test the base of the triaxial cell, or any other element with known dimensions, is used to establish the scale of all measurements taken by a theodolite in a particular position.

2. Wolfskill and Buchanan (47) used an etched girdle band at the center height of the specimen and measured with an optical microscope. Only the mid-height of the sample was measured.

Werner (43) measured the lateral deformation at the mid-height of a triaxial test specimen under repetitive loading. A 3/8-inch steel band cut from a clock spring, was machine scribed to 1/100-inch markings at one end. At the other end a vernier was scribed to make possible an accuracy of 1/1000 inch in the observed measurements. The band was placed around the circumference of the specimen, at its middle height, and was constructed to overlap with the scale on one side and the vernier on the other which were read by a short focal length telescope clamped to the triaxial device. The band was lubricated, as well as the sample, and rubber bands were placed around the metal band to prevent slippage. Werner states that the lateral strain at the mid-height of a triaxial specimen would be representative of field conditions. He also admits that the band measuring to 0.001 inch was not the precision instrument it was hoped to be and that there was doubt as to whether the band was fully in contact with the specimen all the time. He also points out the possibility of friction between the band and the membrane, and the constriction of the specimen by the steel and rubber bands.

Folque (13) measured lateral deformations in triaxial tests using a steel wire anchored at one end and connected at the other to a mechanical strain-gage. The steel wire was wound around the sample over small steel rollers. He does not give any details of this method of measurement in the listed reference.

3. Measurement of lateral deformations of triaxial test specimens using electrical methods has been accomplished by some investigators (14, 10). In reference (10) a test device which permits the determination of horizontal deformations of cylindrical samples by means of electrical capacitive methods is given. Reference (14) mentions that the axial and lateral deformations of triaxial test specimens were made electrically and that they could be recorded photographically using a multi-channel high speed galvanometer recorder, but there are not enough details given.

4. In vacuum triaxial tests on dry sand, the author measured the lateral deformation at the mid-height of the specimen using four strain gauges placed so that each two would be facing each other along the same line, and being perpendicular to the other two. The details of these measurements will be given in Chapter 4. This method was also used by the author in the case of unconfined compression tests on clay.

5. The author suggests measuring lateral deformations of triaxial test specimens by taking photographs of the entire specimen at various strain levels during the test. It is recommended that two cameras be placed at known distances, closest to the specimen and in such a way that photographs of the specimen in two perpendicular directions could be taken. The scale in each photograph can be determined by including a subject with known dimensions such as the base on which the specimen rests. To get higher accuracy the photograph can be projected on a wide calibrated screen, that is magnified several times. The advantage of this method is that the entire profile of the specimen is shown and its cross-section at any height can be measured. This gives a measure of volume changes and an indication of the uniformity of deformations. The method has been tried by the author, using one camera only, in a very limited number of triaxial tests on dry sand. The indication was that this method is very time consuming and expensive. More work has to be done before any final conclusions can be drawn as to the exact degree of accuracy of this method and its usefulness.

The preliminary design of a device intended to register axial and radial deformations of a triaxial soil specimen is given in reference (25). A parabolic mirror with a bulb at the focus, both placed inside the triaxial cell with air pressure, sends a bundle of parallel rays toward the specimen.

The shadow of the specimen is projected on a photographic film mounted on a frame that can be rotated from outside the chamber through a set of gears.

Shocklay and Ahlvin (32) took full photographs of vacuum triaxial test specimens of sand, at certain axial strains, to determine the average diameter and apparent volume changes.

6. The average cross-sectional area of a saturated triaxial test sample, in drained triaxial tests, may be obtained by determining the volume change and decrease in height of the sample, at any strain level, and assuming that the cylindrical shape is retained throughout the test.

7. Bishop and Henkel (3) give a device that measures the change in the circumference of a triaxial test specimen at one height. The lateral deformation indicator is placed inside the pressure chamber in contact with the specimen.

8. In undrained triaxial tests and unconfined compression tests on saturated clays, the assumption of no volume change taking place during the test can be made. The cross-sectional area of the specimen at any instant during the test (A) can thus be determined knowing the original cross-sectional area (A_1), the initial height H_1 and the change in height ΔH at that moment. Assuming constant volume and cylindrical shape during the test (9) the following relation can be obtained:

$$A_1 \; H_1 = A \; (H_1 - \Delta H)$$

or

$$A = \frac{A_1}{1-\epsilon} \quad \ldots\ldots\ldots\ldots \quad (4)$$

where

ϵ = the axial strain at that instant.

A plane-strain apparatus is believed to be more suited and superior to the triaxial test equipment in the measurement and study of lateral deformations in soils. Such an apparatus, however, is still not available in any final, acceptable, and practical form.

The indirect determination of the lateral strain ratio will be briefly discussed in the following paragraphs. Most of these methods are based on assumptions which may not always be absolutely correct in the case of soils. The following methods represent some possibilities for the indirect determination of the lateral strain ratio for soils:

1. Consider the element of soil shown in Fig 2 (p. 9) subjected to a vertical stress σ_z. The shown horizontal stresses σ_x and σ_y, which are assumed to be equal due to symmetry, are produced due to the confinement of the prism (21).

A stress σ_z acting on one side of the prism produces strains in its own direction z and in the two perpendicular directions x and y. Based on the theory of elasticity, thehorizontal strains are equal to the vertical strain multiplied by Poisson's ratio μ'. For no lateral strains to occur in any horizontal direction (that is, the condition of earth pressure at rest) stresses must be added in the horizontal direction, opposite to the existing stress, that would make the strain in this direction equal to zero (as shown by the dotted arrows in Fig 2 for the x-direction). In this case:

$$\sigma_x - \mu'\sigma_z - \mu'\sigma_y = 0$$

$$\sigma_x (1-\mu') - \mu'\sigma_z = 0$$

$$\frac{\sigma_x}{\sigma_z} = \frac{\mu'}{1-\mu'}$$

The coefficient of earth pressure at rest k_o is defined as the ratio of the horizontal to vertical pressures when no lateral movement occurs. Then:

$$k_o = \frac{\mu'}{1-\mu'} \quad \ldots \ldots \ldots \ldots \ldots \ldots \quad (5)$$

This formula was developed by Terzaghi (40) and is presented here to show the assumptions used. Based on this formula the lateral strain ratio for soils can be determined if the value of the coefficient of lateral earth pressure at rest for the soil k_o is obtained by some means.

Terzaghi (39) evaluated the coefficient of lateral earth pressure at rest by determining the value of the lateral pressure exerted by a soil sample on the walls of a container.

Another method suggested by Krynine (21) for determining the coefficient k_o is to enclose a sample in a rubber envelope placed in a container with water. The sample is subjected to loading and the pressure transmitted to the water is measured (which is the lateral pressure of the soil).

In references (41, 42) a ring type device, similar to that used in consolidation tests, was used. Loads were imposed upon a compacted sample of sand confined in the ring. Strains were measured in the ring by SR-4 strain gages attached to the ring. By varying the degree of restraint, through changing the thickness of the ring, it was hoped to get values of k_o by extrapolation (k_o being the value of the coefficient of lateral earth pressure when the strain equals zero). The tests were considered inconclusive.

Bishop and Henkel (3) describe what they call the k_o-triaxial test where a vertical load is applied to the soil specimen and the lateral pressure varied until there is no lateral deformation in the specimen. The lateral deformation is determined using the indicator mentioned before, and described in the same reference.

In vacuum triaxial tests on dry sand, the author suggests the use of the k_o-test idea given below. The lateral deformation in this case can be checked by strain gages touching the specimen.

In the preceeding paragraphs various suggestions have been presented for the determination of the coefficient of lateral earth pressure at rest. Experimental evaluation of these methods is required before any final conclusions can be reached.

2. Using the assumptions made by the theory of elasticity, formula number 2 can be derived, giving the relation between the modulus of elasticity E', Poisson's ratio μ' and the modulus of rigidity G'. This formula can be used to evaluate the lateral strain ratio for soils if the modulus of deformation, and the modulus of shear deformation can be determined. This is a problem that still remains to be solved and the variation of these so-called constant quantities (E', G', and μ') for soils under various conditions has to be studied.

3. The volumetric strain $\frac{\Delta V}{V}$ of an elastic element subjected to triaxial loading is given by formula number 3 (with $\sigma_2 = \sigma_3$). In a drained triaxial test, the volume change taking place can be measured throughout the test. If the modulus of deformation is determined for the soil in question, then equation 3 can be used to evaluate the lateral strain ratio for the soil.

The methods previously described for the indirect determination of the lateral strain ratio for soils need to be tested experimentally to determine their validity and applicability for different soils and varying conditions. The pattern of variations of μ , if any, must also be determined.

3.5 Laboratory Testing Used in the Determination of the Stress-Strain Characteristics of Soils.

There are many tests used to evaluate the stress-strain properties of soils. Most of these tests have been mentioned in the preceding parts. All available equipment at the present time have some shortcomings as far as the determination of stress-strain behavior of soils is concerned. Some modification of present day testing equipment and procedures, or the development of entirely new tests, is needed.

Since many practical problems in soils mechanics approximate more closely the condition of plane strain than that of axial symmetry, used in the triaxial test, and due to the difficulty of measuring lateral deformations of a triaxial test specimen, the author tried to develop a plane strain apparatus with controlled drainage, or a triaxial test that permits independent control of the three principal stresses, so that generalized states of stress can be examined, including the important case of plane strain. However, the relatively high compressibility of the soil skeleton and the magnitude of strains required to cause failure lead to mechanical difficulties which make independent control too complicated. Both ideas were abandoned after realizing the many practical problems involved and the long period of time required to arrive at any reasonable design, if possible. The plane strain apparatus described by Conforth (8) is to be considered as only one trial in the right direction, since it still has many limitations. The continuation of this and similar attempts, to reach a final design of plane strain as well as other equipment in this area, is strongly urged.

The triaxial test is still the most popular test in soil mechanics used to determine strength and stress-strain properties of soils. It has advantages and limitations that will be presented later on. In spite of its disadvantages, the triaxial test is considered by the author to be the best available test at the present time, especially if some modification could be made to overcome its shortcomings (as will be discussed). This conclusion was reached after a thorough review of the literature concerning testing of soils. The triaxial test is extensively used by various investigators. The author believes that to obtain the stress-deformation properties of soils for the varying conditions encountered in the field, more than one type of test would probably be required. Test conditions must reproduce, as closely as possible, field conditions of the problem at hand.

Since it has been concluded that the triaxial test is the most widely accepted test, at present, and since it is the test used in the laboratory study presented in the next chapters, a discussion of the advantages and limitations of this test appear to be worthwhile at this point. The triaxial test has many advantages among which are:

1. The stress conditions, although not absolutely constant throughout the sample, are nearer to constant than in other types of apparatus. Also all stress values are known with fair accuracy throughout the test (37).

2. Control of drainage conditions during the test is possible.

3. Determination of volume changes and pore pressures of saturated soil specimens can be accomplished throughout the test.

4. Test results are analyzed and explained by the conventional Mohr theory.

5. Various specimen sizes can be adapted depending on the grain size distribution of the soil. Different types and rates of loading are also possible.

6. Confining all-round pressure is controlled in the triaxial cell.

The following are the main disadvantages of the cylindrical triaxial test:

1. In conventional triaxial tests soil specimens are consolidated under equal all-round pressure, that is, isotropic consolidation. This is not believed to be the case, however, in the field where anisotropic consolidation occurs. The initial state of stress in nature, before any change takes place, must be correctly represented in the test. Results obtained from tests, in which the initial consolidation is under an equal all-round pressure, cannot therefore be applied directly to practical problems without making allowance for the probable stress ratio in the natural ground (3).

2. One of the main criticisms of the triaxial test is the nonuniformity of stresses and deformations at all but extremely small strains due to friction at the end platens which cause barrelling effects (30). Cylindrical triaxial test samples are subjected at their top and bottom surfaces to friction which prevents lateral strain and causes bulging to occur at the center of the sample, which indicates that conditions are not uniform (37).

Research into stress-strain relationships of soils requires a method of applying a controlled set of stresses. The triaxial test approaches this requirement at small strains. However, at large strains, non-uniformity of sample deformation is the source of numerous errors. In undrained tests, non-uniformity of stress causes pore pressure gradients and local drainage within the soil specimen, depending upon the speed of testing (30). There is a possibility that even at unchanged water content there is a redistribution of water content within a triaxial test sample which may change its strength characteristics in a manner not necessarily representative of a large soil mass (7). Thus pore water pressure measurement at the base of a triaxial specimen cannot be applied to calculate effective stresses at the center. Non-uniform density and volume changes, throughout the triaxial compression test specimen during loading between rigid end plates, are reported by Shocklay and Ahlvin (32). They state that if non-uniform conditions exist in a triaxial test specimen, then stresses, strains, and volume changes computed on the basis of average specimen conditions may not be at all indicative of the changes that are occurring in the failure zone, especially in cases where failure occurs as bulging of the specimen rather than on a well-defined failure plane.

Werner (43) states that the behavior of the triaxial specimen as a whole would not approximate that of the elemental volume due to the effect of friction at the end plates. Employing the triaxial method to approximate field conditions, the lateral deformation at the mid-height of the specimen would be representative of field conditions as it is farthest from the end restraint.

There is a stress variation from the center of the specimen to its periphery, but if the triaxial test specimen is chosen with a length to diameter-ratio of from two to three, an accurate shear plane is developed which is unaffected by frictional end effects. This will minimize the effect of end restrain on the soil strength (23).

Taylor (37) concludes that in spite of the non-uniformity in the triaxial test, the average action of the sample in the test is the only action that can be considered since it is the nearest obtainable approach to the true stress-strain condition of the soil in the field.

3. The ideal laboratory test for soils would be one with uniform stress and strain throughout the sample. Actually such a condition is not possible, since non-uniform conditions of stress and strain occur in all types of tests, and failures are always progressive, to a degree (37). It should be remembered, however, that if the loading on a large mass of soil in nature is gradually increased until failure is incipient, the stresses will seldom, if ever, be uniform over an entire rupture surface, and failure will not be reached at all points at the same time, but will be progressive. The strains that occur will not be uniform, and concentrations of stress will tend to take place at points of maximum strain, causing rupture to start at these points and then progress through the mass (37).

4. The axial load is reduced by piston friction (3). The friction between the loading rod and the bushing may become appreciable, especially due to lateral movement that occurs sometimes at or before failure.

5. A factor which is not reproduced in the conventional triaxial test, and which occurs in many practical problems, is the rotation of the planes of principal stresses during the test (3). Broms and Casbarian (6) found that this rotation, and the intermediate principal stress, had some effect on the deviator stress, the angle of internal friction, and the pore water pressure of a remolded clay which they tested.

The lack of separate control of each of the three principal stresses is definitely a disadvantage of the triaxial test. In the conventional triaxial test the intermediate and minor principal stresses are equal, constant during the test, and are controlled together. Henkel (18) states that the conventional triaxial tests represent conditions where no change in the intermediate and minor principal stresses accompanies the shearing process, and thus are of limited direct application.

In a triaxial test specimen the strains are larger than those that would occur had the specimen been confined by like soil material. The triaxial confinement does not produce the lateral strains and passive resistance occurring in nature. The question of representative lateral support then becomes a question of whether or not the lateral pressure should be increased as the axial strain increases, and, of course, in what proportion (47).

6. There are many factors that affect the measurement of pore pressures in a triaxial test. One of these factors is the flexibility of the measuring system (44). The authors give a new measuring system which has a very rapid response time and which uses electrical pressure transducers in the measurement (44).

In long duration tests, leakage through membranes, past the bindings of membranes to the cap and base, and from valves, fittings, and seran tubing, becomes a problem. Poulos (27) gives various recommendations to minimize this effect and concludes that leakage may only be important in very long duration tests.

In long duration triaxial tests, the change in the cell pressure is also an important factor to be considered. Bishop and Henkel (2) describe a self-compensating mercury control that accurately maintains a constant cell pressure during tests of long duration.

Another factor that affects the results of triaxial tests is the effect of the rubber membranes and filter paper drains used to accelerate consolidation. Henkel and Gilbert (19) showed that the rubber membrane and paper drains gave rise to an apparent increase in strength that is only dependent on the stiffness of the membrane and paper. Bishop and Henkel (3) say that due to restraints imposed on the specimen by the rubber membrane enclosing it and the filter paper drainage strips, a correction to the measured strength has to be made (which is given in this reference).

Air trapped between the sample and the rubber membrane can be dissolved in the pore water under high cell pressures and may affect the test results.

7. The condition of axial symmetry used in the triaxial test is not the only case encountered in nature. Strain conditions affect the test results (8).

8. It is difficult to determine lateral deformations in the triaxial test.

Many of the shortcomings of the triaxial test can be eliminated by applying the measures suggested by the various investigators in this field, some of which have already been mentioned in the preceding paragraphs. The following are some other suggestions along the same line:

(A) The friction of the loading ram can be avoided if the load acting directly on the specimen is measured. Sparrow and Beaty (34) suggest that this be done by using a built-in electrical axial load cell, in the form of an electrical transducer feeding potentiometer or galvanometer recorders which would enable the automatic recording of the axial load, with no friction, for tests with varying durations. Using Thompson ball-bushing will also reduce the friction of the loading rod. The author, in Art. 4.3 describes a calibration method he used in the laboratory study for this purpose.

(B) The end restraint is, in the author's opinion, the most serious problem of the triaxial test, and one that has not yet been solved. Rowe and Barden (30) suggest the reduction of end friction by using a polished aluminum alloy end platen coated with a thin smear of silicone grease and separated from the soil sample by a circular rubber membrane. The end plates were larger in diameter than the soil specimen, in order to accomodate the anticipated lateral expansion. It is claimed that this method allows the use of shorter test samples.

(C) Triaxial tests, where the confining pressure is varied as a certain ratio of the axial stress to which the specimen is subjected, may be utilized (anisotropic consolidation). The change in the direction of the principal stresses may also be accomplished (6).

(D) The leakage from the system can be minimized in long duration triaxial tests if Poulos' recommendations (27) are followed. Among these recommendations are the following:

(i) Two membranes should be used with a layer of silicone grease in between.

(ii) O-ring bindings are suitable so long as the cap and base are polished and greased before putting on the membrane and rings. (The author found that one thicker O-ring gives a better seal than a number of thinner ones having the same diameter.)

(iii) Most valves and fittings in general use in the drainage system of triaxial cells are satisfactory, but their number should be held to a minimum.

The preceding modifications suggested in the triaxial test are believed to eliminate many, though not all, of its disadvantages. More should be done in this respect to make the conditions of triaxial test specimens representative, as closely as possible, of those in nature. At the present time, and until such state is reached or new testing equipment and procedures are arrived at, the triaxial test should be used together with other tests, keeping their limitations in mind.

PART B

LABORATORY STUDY

CHAPTER IV

VACUUM TRIAXIAL COMPRESSION TESTS ON DRY SAND

4.1 General

The importance of a laboratory study of stress-strain properties of soils has been pointed out in several parts of Chaps. I, II, and III. In spite of the lack of an ideal test that can be used to perform such studies, the conclusion has been reached that the triaxial test may be utilized to do a good part of the job at the present time. The many factors believed to influence stress-deformation characteristics of soils have been presented in Art. 3.2. Laboratory studies should be performed to investigate these variables and to find out the behavior of different soils under the various conditions encountered in the field. Very few studies of this type have been carried out, and most of the factors outlined in Art. 3.2 remain to be studied. The need for more research along this line is urgent.

The laboratory study presented here is designed to cover some of the factors affecting the stress-strain behavior of soils and to add some more to our present insufficient knowledge in this complicated and important subject. Any such study is bound to cover only some of the many variables involved. Nevertheless, these limited studies, as they accumulate, are believed to be very helpful and valuable in forming the overall picture.

In the present investigation, quick, or undrained, triaxial tests were used. Only one-cycle static loading was applied to the soil specimens throughout the study.

This chapter covers vacuum triaxial tests on dry Colorado River sand. Lateral deformation of the specimens was measured during these tests. A study of the stress-strain properties of the sand was made. The axial, as well as lateral, strain characteristics were investigated. The study also included strength properties, and volume change during shear, of the sand. The behavior of the sand was investigated under varying density conditions that covered the entire range of densities attainable with this soil. The confining pressure was varied within a small range, representing shallow depths, due to limitation in the amount of vacuum that could be obtained in the laboratory. The rate of strain, or loading, was also changed within the limits believed to be normally encountered in laboratory testing to see the effect on the properties of the sand.

4.2 Soil Used

The soil selected was a clean, dry sand known locally as Colorado River sand. This is a light brown sand obtained from the banks of the Colorado River at Austin, Texas. When examined under a magnifying lens, the sand particles were found to be rather subangular in shape and had a somewhat rough texture. Mineralogically, the sand grains were primarily quartz, with some fragments of igneous, metamorphic and sedimentary rocks. Moreover, the sand was found to be quite rich in silica.

The sand was air dried, then sifted on a "Rotex" sifter style No. 12, with a pulley speed of 520 to 560 rpm. The sifter had two U. S. Standard sieves, No. 20 and No. 200. The output, passing sieve No. 20 but retained on sieve No. 200, was collected for use in this study. The sand was kept at room conditions of temperature and humidity. The hygroscopic moisture content of the sand was 0.12%.

A sieve analvsis was run to determine the grain size distribution of the sand. The results of the mechanical analysis are shown on a semilogarithmic plot in Fig 3, together with grain size distribution curves of the

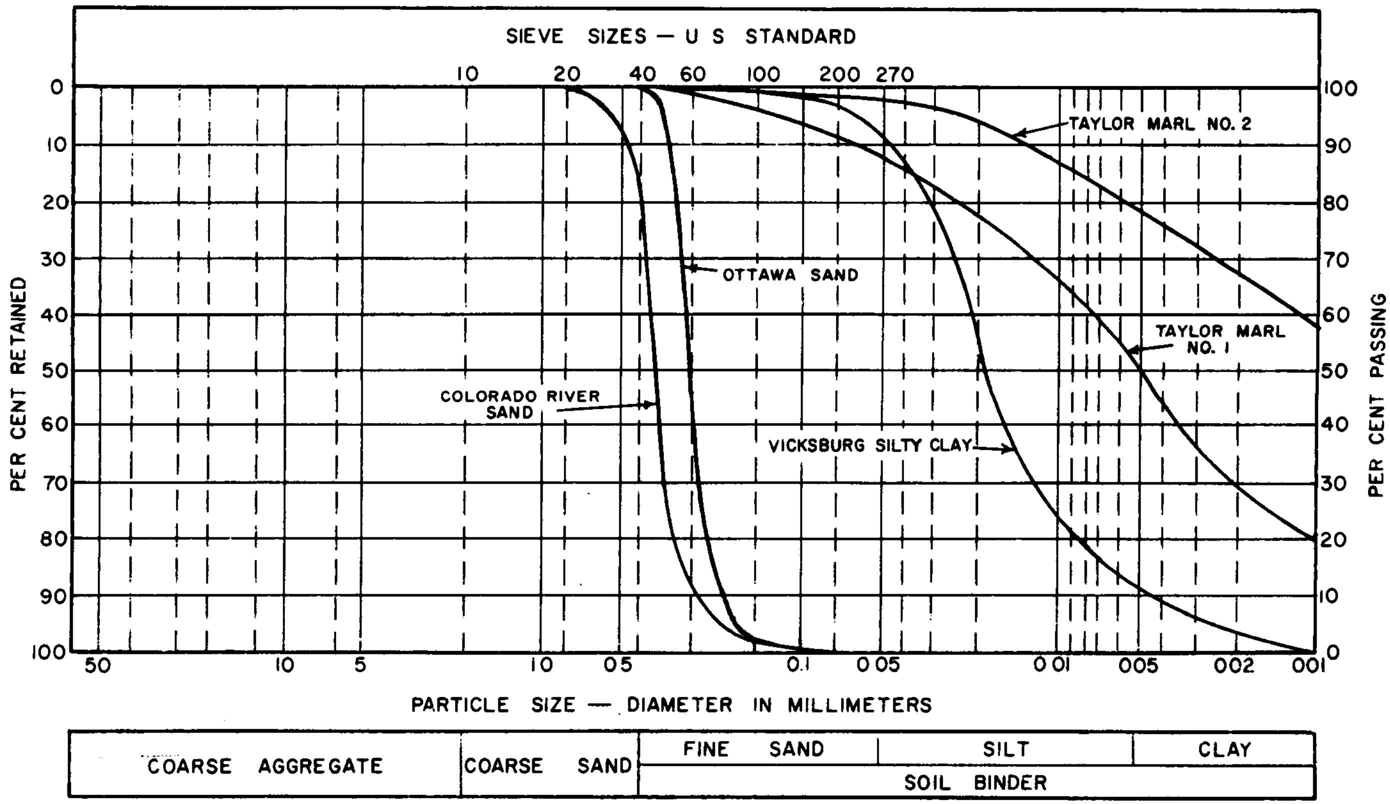

FIG. 3. MECHANICAL ANALYSIS
GRAIN SIZE ACCUMULATIVE CURVES

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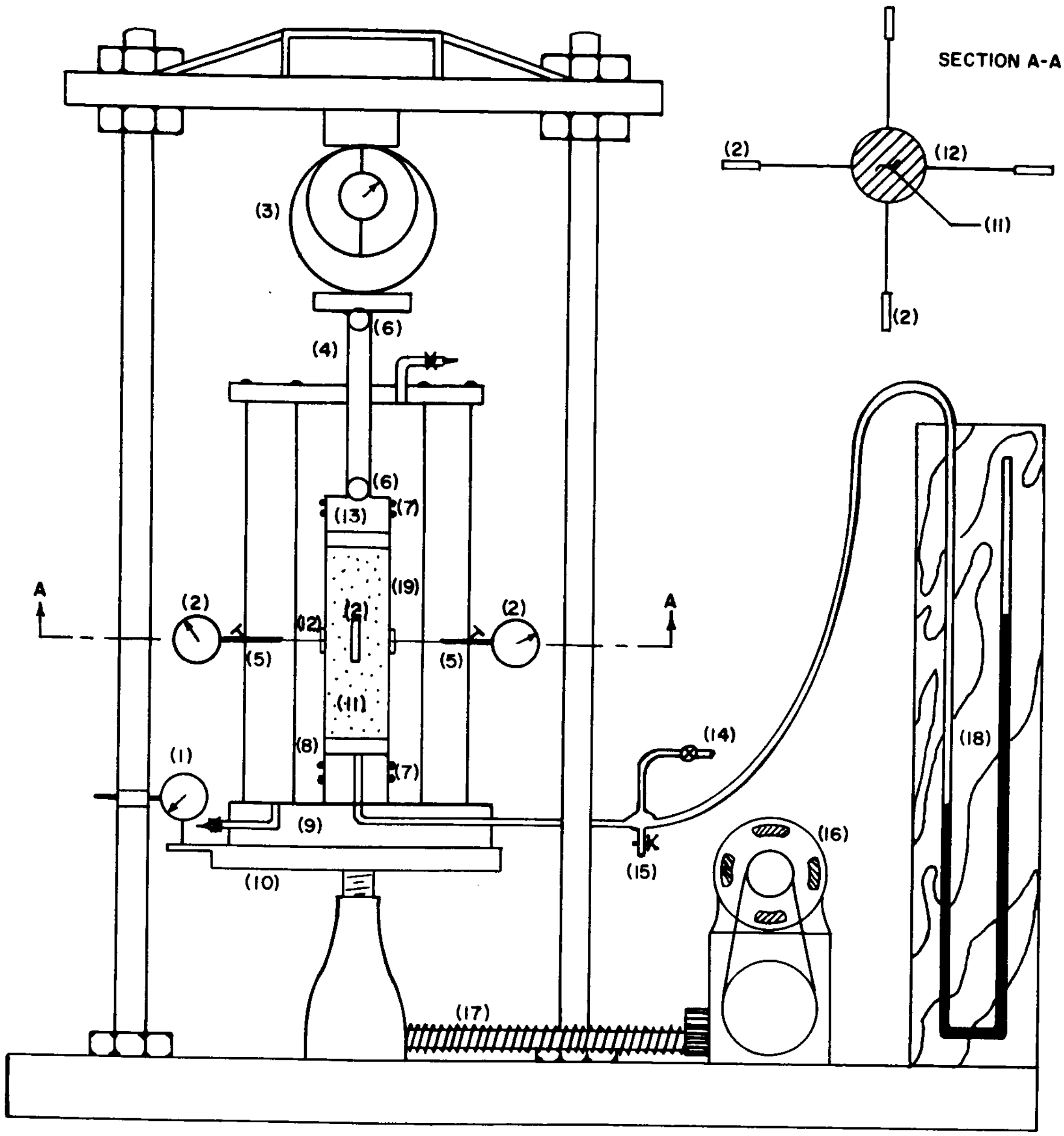
other soils investigated. The resulting curve shows that the soil used can be classified as a uniform fine sand.

The specific gravity of the sand used was found to be 2.67. Both specific gravity and sieve analysis tests were performed in accordance with usual engineering practice and local procedures at the soil mechanics laboratory of The University of Texas (9).

The sand was tested at three different densities. The highest density used was 108.26 p.c.f. which was near the maximum that could be attained. The lowest was 94 p.c.f. which was almost the minimum density that could be obtained with the sand being tested. The third density was in between and equal to 102 p.c.f. to represent the medium density condition.

4.3 Testing Equipment

The triaxial cell used in this study was a Clockhouse cell model T150 (manufactured by Clockhouse Engineering Ltd., England). The lucite cylinder, forming the pressure chamber, was removed and the base of the cell was fitted so that it could be connected to the vacuum source. This connection could either be opened or closed, using the valve at the base of the cell, and it formed one branch of a three-way outlet from the vacuum source. The second branch led to a bleeding valve, which together with the one used to open and close the vacuum source, could be utilized to control the amount of vacuum to which the sample was to be subjected. This vacuum acting on the sand specimen through its base, represented the all-round confinement σ_3 , analogous to the lateral pressure in the conventional triaxial test. The third branch of the three-way outlet was connected to a mercury manometer measuring the vacuum which the sample was subjected to. The test set-up is shown in Fig. 4.



(1) AXIAL DEFORMATION DIAL
(2) LATERAL DEFORMATION DIALS
(3) PROVING RING
(4) LOADING ROD
(5) CLAMPS FIXED TO THE TRIAXIAL CELL ROD
(6) STEEL BALL
(7) O - RINGS
(8) POROUS STONE
(9) BASE OF THE TRIAXIAL CELL
(10) MOVEABLE PLATFORM
(11) TEST SPECIMEN
(12) METAL DISC AT THE END OF THE LATERAL DEFORMATION DIAL STEM
(13) UPPER CAP AND PLATE
(14) VACUUM SOURCE AND REGULATING VALVE
(15) BLEEDING VALVE
(16) MOTOR AND PULLEYS WITH SPEED CONTROL
(17) SHAFT
(18) MERCURY MANOMETER
(19) RUBBER MEMBRAME

FIG. 4. TEST SET-UP

The lateral deformations at the mid-height of the test specimen were measured using four dial extensometers touching the circumference of the specimen and rigidly attached by clamps to the steel rods forming the outer part of the cell, as shown in Fig. 4. The fixed clamp holders are cast-iron clamps purchased from Curtin Company in Houston, Texas (model No. 4113). The extensometers are Ames 312.5 Jewelled with a 1/2-in. travel, purchased from B. C. Ames Company, Waltham, Massachusetts. These extensometers were fitted at the end of their stems with a 1/2-in. diameter brass disc of 1/16-in. thickness with which to bear on the sample. These discs were used to prevent any concentration of stresses at the points of contact between the specimen and the stems of the extensometers. The sensitivity of the extensometers was 0.0001 in. The four dial gauges were placed so that each two would be facing each other along the same straight line, to measure one diameter at the mid-height of the specimen, and at the same time be perpendicular to the other two, as shown in Fig 4. This means that two perpendicular diameters were measured at the middle portion of the sample during the test. The axial deformation of the sample was measured by an extensometer of the same type but which had a travel of 2 in. and a sensitivity of 0.001 in. It was firmly attached to the loading machine as shown in Fig 4.

The loading machine used was of the constant-rate-of-strain type (manufactured in the laboratory using a screw jack from the Duff Norton Manufacturing Company, Pittsburg, Pennsylvania). The machine is motor operated with a gear to control the rate of loading. Four speeds of loading were used in this investigation. These were 0.02, 0.04, 0.08 and 0.16 in. per minute, which corresponded to rates of strain of 0.625, 1.25, 2.5, and 5% per minute respectively (with a 3.2 in. height sample). The applied load was measured by a double proving ring attachment (model no. 2126, Soiltest, Chicago), fitted to the upper part of the machine, as shown in Fig. 4. The accuracy of load measurement was 0.08333 lb.

The friction of the loading rod of the triaxial cell was checked by means of a calibrated load cell. The load cell was placed in the position of the specimen and loaded through the rod, using the machine described above. Readings of the proving ring, measuring the load at the top of the loading rod including any friction, and of the calibrated load cell, measuring the load at the tip of the rod excluding friction, were taken simultaneously throughout the entire range of loads that could be measured by the ring. These two readings were then compared to determine the amount of friction, if any, at various load levels. The two readings were almost the same throughout the entire range. The tendency was, however, for the friction to increase as the load was increased, but the maximum friction value observed, that is the maximum difference between the two readings, was less than 1.0% for the ultimate load that could be obtained with the proving ring used. The friction was much less, almost zero actually, for the lower loads encountered in the laboratory investigation. To be sure that the measured friction was the real value, the calibrated load cell was loaded directly by the proving ring and both readings again taken. Both readings were found to be almost exactly the same indicating that both the ring and calibrated cell are correct. Thus it was concluded that for all practical purposes of this investigation the rod friction can be neglected as it did not amount to any appreciable value.

It should be pointed out that the above procedure may be used to eliminate the effect of friction of the loading rod in the triaxial test. After determining the amount of friction at various load levels, as described before the net load during the test can be obtained by subtracting the friction from the measured load. This is another way to take care of this particular limitation in the triaxial test, besides the methods described in Art. 3.5. In conventional triaxial tests, the friction of the rod can be checked, while the cell pressure is applied, by using a proving ring placed inside the triaxial chamber (instead of the calibrated cell.)

4.4 Preparation of Test Specimens

The equipment used in sample preparation consists of the base of the triaxial cell and the cap, a thin rubber membrane of the proper size, a small vibrating platform, a tamping rod, a funnel, a forming jacket (split mold) and O-rings and porous stones of the proper size. The forming jacket consisted of a cylindrical tube, split longitudinally in two halves which can be fastened together by C-clamps. The split mold was lined from the inside with a wire mesh, and the inside clear diameter of the mold was 1.44 in. with a height of 5.25 in. The forming jacket also had two opposite holes through which vacuum could be applied and distributed through the wire mesh.

The prepared sand specimens were 1.4 in. in diameter and 3.2 in. high. To prepare a specimen having a certain density, the weight of sand necessary to produce that specimen, with the required dimensions, was determined. This weight of air-dried sand was then divided into eight equal parts with each part representing one layer. The sample was prepared in eight layers to assure uniform density conditions throughout.

The rubber membrane (0.021 in. thickness) was placed around the base of the triaxial cell and the bottom porous stone and tied tightly with two O-rings. The two halves of the forming mold were placed around the rubber membrane, while holding the loose end of the membrane vertically upward, then fastened tightly together using the C-clamps. The upper end of the rubber membrane was turned down over the edge and on the outside of the forming jacket. The membrane was made to fit flush against the inside surface of the mold by attaching a vacuum to the external ports of the jacket and applying longitudinal tension to the membrane, smoothing it to a cylindrical surface, before any soil was placed in the mold.

The next step in the preparation of the sand specimen depended on the density to be achieved. Three densities were required, as mentioned in Art. 4.2. The specimens having maximum density were prepared by placing the base, with the forming jacket assembly described above, on a small vibrating platform (made by J. Yates Mfg. Company, Chicago). The sand was then placed into the mold in eight equal layers, that had already been weighed and prepared as described before. Each layer was placed through a funnel at a constant height (6.0 in.) which was moved around while the sand was being poured so that the entire cross-section of the mold might be covered uniformly with the sand. Each layer was then vibrated and tamped with a rod for a few seconds. The vibration time, for the eight layers starting from the bottom, was 25, 30, 35, 40, 45, 50, 55, and 60 seconds respectively (this time was arrived at by trial). The blows for each layer were distributed uniformly over the area of the specimen. The number of blows was more for the upper layers and decreased regularly to the bottom. The specimens with the medium density were prepared in the same manner except that no tamping was applied in sample preparation, only vibration, and that the vibration time in this case was 3, 4, 5, 6, 7, 8, 9, and 10 seconds for the eight layers respectively, from the bottom up. The specimens with the lowest density were also prepared in the same way except that no rodding or vibration was applied, that is, the samples were prepared by pouring sand into the mold from a fixed height (6.0 in.). This method is believed to yield specimens that have uniform density throughout.

The next step was to level the top of the sand specimen, as much as possible, using the tamping rod. The upper smooth lucite plate was then put on the leveled sand surface and rotated slightly to obtain good seating. The upper head was placed and the membrane rolled up around it and securely tied to the cap by O-rings. The upper head was aligned to a horizontal position allowing as little movement as possible. Vacuum was applied to the specimen through its base and cut off from the jacket. The applied vacuum was then increased to the desired testing pressure and the forming jacket removed.

Measurements of the specimen were then made. Several measurements of the diameter at the top, middle and bottom of the sample were taken. The height of the specimen at four perpendicular positions was also measured. The average of the several readings was considered to be the dimensions of the specimen. All measurements were made to an accuracy of 0.01 in. In getting the diameter of the specimen, the diameter plus the rubber membrane thickness were measured and then the thickness of the membrane subtracted from the reading to give the actual diameter of the sample. The height was determined by measuring the total height of the specimen, porous stone, base, upper lucite disc, and cap. Knowing the thickness of all other items, the height of the specimen could be determined. If the diameter of the specimen was not 1.4 in. or its height not 3.2 in., the specimen was rejected. After sample preparation, the specimen was not moved any more than necessary.

As described above, the specimen density and dimensions were strictly controlled in all tests. This was done by determining the weight of sand that would give a cylindrical sample 1.4 in. diameter and 3.2 in. high and which would have a certain density (weight = volume times density) then forming the specimen with this amount of sand and checking its dimensions. It should be noted that the length-to-diameter ratio of the specimens was 2.286, (which agrees with Taylor's recommendations as described in Art. 2.5).

4.5 Test Procedure

The object of these tests is to determine the stress-deformation and strength-characteristics of dry Colorado River sand subjected to shearing stresses produced by varying the principal stresses being produced by the application of a vacuum to the sand specimen through its base.

After the specimen was prepared, as described in Art. 4.4, the cell was put together and carefully placed on the loading machine, with the sample inside. The four gauges, which measured lateral deformations, were than fastened in place, as shown in Art. 4.3. The loading rod was lowered to touch the ball on the top of upper cap that rests on the sand specimen. The loading head of the testing machine, with a proving ring attached to it, was brought down till the lower part of the proving ring came in contact with the steel ball on top of the loading rod. The proving ring, axial deformation dial, and four lateral deformation extensometers, were then set to zero, after checking the mercury manometer to make sure that the exact vacuum was acting on the sample. The loading was then started and the vacuum kept constant during the entire test.

During loading, readings of the proving ring and the five extensometers were taken every 0.01 in. of axial deformation. Loading was continued until failure was reached and the specimen bulged to a great extent and, or, tilted. The loading was then stopped and a sketch of the failed specimen was made. The vacuum was disconnected and the apparatus dismantled.

The testing program was designed to include 48 tests. One third of this number had specimens of 108.26 lb/ft^3 density, the second third of the specimens had a density of 102 lb/ft^3, and the specimens in the remaining 16 tests had a density of 94 lb/ft^3. For each density 4 tests were run at a rate of strain of 0.625% per minute, 4 other at 1.25% per minute, 4 more at 2.5% per minute, and the last 4 at a rate of strain of 5% per minute. For each density and rate of strain, specimens were tested at the following four vacuums, 2.32, 4.64, 6.95, and 8.69 psi, which corresponded to 12, 24, 36, and 45 cms of mercury respectively. Each one of these 48 tests was repeated twice and the results of both tests averaged. This average was then considered in the final analysis.

4.6 Results and Discussion

In this article a presentation of the test results will be made. Also a discussion of these results and their significance will be presented. The assumptions made will be given and their degree of accuracy considered.

A. Computation of Stresses and Strains

The axial load acting on the test specimen at any time, in excess of the confining pressure σ_3 , was obtained from readings of the proving ring. This load will be referred to here as P . The deviator stress σ_Δ is computed by dividing this load by the area of the specimen A at that time. It is defined as the difference between the major and minor principal stresses, that is $\sigma_1 - \sigma_3$.

The area of the specimen at various strain levels, used to compute the deviator stress, was obtained from readings of the lateral deformation dials at the mid-height of the specimen. Photographs of the specimen at various strain levels revealed the fact that the specimen does not deform laterally at its ends because of the friction developed there between the sample and the plates, that is, end restraint. The photographs also showed that the maximum lateral deformation of the specimen was about its mid-height. Thus the assumption was made that the average lateral deformation of the sample would be the average of the maximum value at its mid-height and the minimum value (zero) at its ends. This means that the average lateral deformation was taken as half the value at the middle of the sample. This assumption is not absolutely correct but is believed to approximate actual conditions.

The lateral deformation of the specimen at its mid-height was taken to be twice the mean reading of the four extensometers measuring such deformation at any instant. The use of the mean value of four perpendicular readings would reduce the error arising from the possibility that the sample when deformed does not remain a perfect cylinder. The average lateral deformation of the specimen Δ_1 would then be equal to this mean reading. The average diameter of the sample at any strain level was obtained by adding the original diameter to the average lateral deformation. Thus at any instant during the test the following quantities could be defined:

Axial deformation	Δ	in.
Axial strain	$\epsilon = \Delta/H_0$	in./in.
(H_0 being the original height of the specimen		in.)
Lateral deformation	$\Delta_1 = (\Delta_{1_1} + \Delta_{1_2} + \Delta_{1_3} + \Delta_{1_4})/4$	in.
Diameter	$D = D_0 + \Delta_1$	in.
(D_0 being the original diameter of the specimen		in.)

Area	$A = \frac{\pi D^2}{4}$	$in.^2$
Lateral strain	$\epsilon_1 = \Delta_1/D_0$	in./in.
Axial load	P	lb.
Deviator stress	$\sigma_\Delta = P/A$	lb./in. in.
Modulus of deformation	$E = \sigma_\Delta/\epsilon$	lb./in. in.
Lateral strain ratio	$\mu = \epsilon_1/\epsilon$	in./in.
Height	$H = H_0 - \Delta$	in.
Volume	$V = AH$	$in.^3$

The above values were determined on the assumption that the sample while deforming remains perfectly cylindrical in shape and that the deformation, both in axial and lateral directions, are uniform and represented by the average values. The average axial deformation is determined directly by an extensometer which measures the axial shortening of the specimen.

As mentioned above, the stresses in this study were determined using areas that were actually determined by lateral deformation measurements. It was hoped that in spite of the far-reaching assumptions made in area computations, the results would be more realistic than those obtained by the conventional method. In the usual triaxial test, on dry sands, areas are determined on the assumption that the cylindrical shape continues and the volume of the specimen remains constant during the entire test, an assumption which is known to be incorrect due to the fact that volume changes and bulging of the sample at its mid-height do occur and become of appreciable magnitude at high strains. Taylor (38) shows that the assumptions made in computing the cross-sectional areas of the triaxial test specimen at any strain, namely constant volume and cylindrical shape, do not introduce objectionable errors up to a strain of 7% for tests on saturated clays. He also stated that this is particularly true for saturated specimens tested under high confining pressures. The difference in results between the conventional method of stress computation and the procedure developed in the present study will be discussed later on.

It must be pointed out here that the results of this investigation showed that the circular cylindrical test sample did not remain so during the entire test. The sample bulged more at its mid-height. Also the cross-section at this level did not remain a perfect circle. In most tests the sample bulged more at two perpendicular sides than it did on the other two, but the two measured diameters were nearly the same.

B. Strength Characteristics

Coulomb gives the shear strength of a soil τ by the formula:

$$\tau = c + \sigma \tan\phi \qquad (6)$$

where

c = cohesion

σ = normal stress

ϕ = angle of internal friction.

For dry clean sands the cohesion is equal to zero. For saturated clays, under conditions of no volume change during shear, ϕ becomes equal to zero (based on the total stress concept).

In discussing the strength of a soil, it is important to specify the criteria used in defining failure and strength. Commonly, in triaxial tests, the strength is measured either by the maximum deviator stress which a specimen will withstand, or by the deviator stress at a given value of axial strain (usually 20% for foundation engineering purposes).

The strength of a dry sand depends on the friction and interlocking between the particles. In such a single grained structure the degree of mobilization of internal friction depends upon the shape, size and texture of the particles, the grading of the sand, the confining pressure, and the movement of the grains.

The angle of internal friction of dry Colorado River sand was determined, for the various conditions of density and rate of strain, using the results of the vacuum triaxial tests interpreted by Mohr's circles. A summary of the results of all tests, as far as the angle of internal friction is concerned, is given in Table 1. The mohr circles and envelopes are given in Appendix B.

Table 1 shows that the angle of internal friction of the sand increases as the density increases. The increase in ϕ with density, however, is more at the lower ranges of density than it is with the higher density values. The reason for this phenomena is believed to be the bigger movement and greater particle interlocking, when the sand is sheared, in the lower density range.

The range of rates of strain which were studied had some effect on the angle of internal friction, but there were no major variations. The interesting observation, though, was that the influence of the rate of strain was greater for dense than for loose sand. However, it was concluded that the use of any rate of strain, within the investigated range, in the laboratory determination of the angle of internal friction for granular soils is permissable. In the present study the maximum variation encountered in the angle of internal friction due to differences in the rate of strain used was in the order of 4 per cent which is allowable. Variation in the results of tests with the same rate of strain are thought to be in the same order.

The specimen in each test was loaded until failure was well developed. In the case of samples with high and medium densities, a well defined failure plane was usually developed for all pressures and rates of strain. This is known as a general shear failure. In the case of specimens that had low densities (loose sand) the failure was either by bulging, generally at low pressures and

TABLE 1

VALUES OF THE ANGLE OF INTERNAL FRICTION (ϕ)

In Degrees At Different Densities

And Strain Rates for Dry

Colorado River Sand

Rate of Strain in per cent per minute	Density 94.00 pcf	Density 102.00 pcf	Density 108.26 pcf
0.625	37.0	45	48.2
1.250	37.2	45.2	49.0
2.500	37.4	45.4	49.6
5.000	37.5	45.5	50.0
Average Value	37.28	45.28	49.20

rates of strain, or sometimes by bulging and very much undefined failure planes for the higher range of pressures and rates of strain. This is known as local shear failure.

C. Axial Stress-Strain Characteristics

The effect of density, confining pressure and rate of strain were studied. It was found that the effect of the limited range of rates of strain investigated was not pronounced. It was therefore decided that only the results of tests with the maximum and minimum studied rates of strain be presented.

The stress-strain curves for dry Colorado River sand are shown in Appendix A. In general the stress-strain curves for the tested sand had an initial part that was approximated by a straight line up to a point, then it curved. The deviator stress increased to a maximum and then decreased again.

The confining pressure had an obvious effect on the stress-strain properties of the sand. The initial linear part of the curve and its slope, the deviator stress at any strain level, and the maximum deviator stress generally increased as the confining pressure got higher. Also the deviator stress decreased more after reaching its maximum value. This pattern was more pronounced for dense than for loose sand conditions.

The density of the sand also had a big effect on the stress-strain curves. The same pattern of changes discussed in the previous paragraph occurred as the density increased and was more pronounced for the higher confining pressures.

The limited range of rates of strain investigated had only little effect. Using any rate of strain, within the investigated range, in laboratory testing would produce only small variations in the results. Although the results are not totally conclusive, it can be stated that the same pattern of variation discussed above may probably develop as the rate of strain is increased. The effects of the rate of strain may generally be expected to be more for higher densities and confining pressures. Although the rate of strain had only minor effects in this study, it is firmly believed that to draw final conclusions in this regard, more tests must be performed at higher rates of strain that ultimately reach the dynamic load condition.

The observations mentioned in the preceding paragraphs will be illustrated by a number of tables that will be presented in the following pages and that summarize the axial stress-strain behavior of the tested sand under the various conditions investigated. It should be pointed out here that there was some deviation in the results of the various tests from the pattern than developed. This is expected due to the various factors that can introduce some error in the results of laboratory determination of stress-strain properties of soils. Among these factors are: (i) variation in density and uniformity conditions from one specimen to the other and within the same specimen, (ii) human errors in recording the various readings taken, (iii) confining effects of the rubber membrane and extensometers measuring lateral deformations, (iv) seating error (see Appendix A), (v) minor changes in confining pressure and speed of loading during the test, and (vi) the measurement of the small deformations at low levels of strain in the very beginning of the test may not be accurate due to the fact that the extensometers used are not sensitive enough for these very small initial deformations (16).

Table 2 gives the axial strain at which the initial portion of the stress-strain curve is no longer a straight line. This strain will be called the initial axial strain and given the symbol ϵ_i . Due to seating errors, no exact trend could be detected. Generally, however, it can be seen that the initial strain decreased as the density and confining pressure was reduced and as the rate of strain was increased.

TABLE 2

Strains at Which the Stress-Strain Curves Cease

To be a Straight Line (ϵ_1) in In./In.

(Dry Colorado River Sand)

Confining Pressure	Rate of Strain 0.625% Per Min.			Rate of Strain 5.000% Per Min.		
(σ_3) psi	Density 94.00 pcf	Density 102.00 pcf	Density 108.26 pcf	Density 94.00 pcf	Density 102.00 pcf	Density 108.26 pcf
8.69	0.0021	0.0026	0.0022	0.0010	0.0017	0.0035
6.95	0.0014	0.0028	0.0033	0.0009	0.0016	0.0030
4.65	0.0000	0.0019	0.0028	0.0000	0.0014	0.0040
2.32	0.0000	0.0007	0.0010	0.0000	0.0009	0.0009

Table 3 shows the axial strain at which the maximum deviator stress occurs (which will be denoted ϵ_{100}). This strain was found to decrease generally as the density was increased. No general trend could be detected for the variation of ϵ_{100} with confining pressure and rate of strain because of the limited range investigated. Generally, however, it would be expected that ϵ_{100} would probably be more for lower confining pressure and rates of strain.

The ratio of $\epsilon_i/\epsilon_{100}$ is presented in Table 4. This ratio generally increased as the density increased, and decreased as the confining pressure and rate of strain increased. The small number of variations from this well developed pattern could be explained by the reasons given earlier.

The maximum deviator stress $\sigma_{\Delta_{max}}$ is given in Table 5. The value of this stress increased with the increase in density, confining pressure, and rate of strain. The increase in stress with rate of strain was more for dense sand and higher confining pressures.

Table 6 shows the ratio between the deviator stress at which the stress-strain curve ceases to be straight line σ_{Δ_i} and the maximum deviator stress. This ratio increased as the density, rate of strain, and confining pressure increased. This indicates that the initial straight line portion of the stress-strain curve increased with the increase in these three variables. The deviator stress at an axial strain of 0.1 in./in. ($\sigma_{\Delta_{0.1}}$), given in Table 7, also generally showed the same pattern of variation mentioned above. The ratio between this stress $\sigma_{\Delta_{0.1}}$ and the maximum deviator stress is presented in Table 8. This ratio seems to be generally decreasing as the density, and confining pressure decreased. The effect of the investigated rates of strain on this ratio was not pronounced, but generally it can be said that the ratio $\sigma_{\Delta_{0.1}}/\sigma_{\Delta_{max}}$ decreased as the rate of strain increased. The pattern of

TABLE 3

Axial Strain at the Maximum Deviator

Stress (ϵ_{100}) in In./In.

(Dry Colorado River Sand)

Confining Pressure	Rate of Strain 0.625% Per Min.			Rate of Strain 5.000% Per Min.		
(σ_3) psi	Density 94.00 pcf	Density 102.00 pcf	Density 108.26 pcf	Density 94.00 pcf	Density 102.00 pcf	Density 108.26 pcf
8.69	0.0773	0.0375	0.0430	0.0960	0.0360	0.0414
6.95	0.0940	0.0400	0.0320	0.0620	0.0260	0.0390
4.65	0.0880	0.0310	0.0340	0.1000	0.0340	0.0340
2.32	0.1000	0.0200	0.0350	0.0090	0.0290	0.0270

TABLE 4

$\epsilon_1/\epsilon_{100}$, Ratios Between the Axial Strain at Which the Stress-Strain Curves Cease to be a Straight Line (ϵ_1) and the Axial Strain at Maximum Deviator Stress (ϵ_{100})

(Dry Colorado River Sand)

Confining Pressure (σ_3) psi	Rate of Strain 0.625% Per Min.			Rate of Strain 5.000% Per Min.		
	Density 94.00 pcf	Density 102.00 pcf	Density 108.26 pcf	Density 94.00 pcf	Density 102.00 pcf	Density 108.26 pcf
8.69	0.0272	0.0693	0.0512	0.0104	0.0472	0.0845
6.95	0.0149	0.0700	0.1031	0.0145	0.0615	0.0769
4.64	0.0000	0.0613	0.0824	0.0000	0.0412	0.0118
2.32	0.0000	0.0350	0.0286	0.0000	0.0310	0.0333

TABLE 5

Maximum Deviator Stress ($\sigma_{\Delta_{max}}$) in PSI

(Dry Colorado River Sand)

Confining Pressure	Rate of Strain 0.625% Per Min.			Rate of Strain 5.000% Per Min.		
(σ_3) psi	Density 94.00 pcf	Density 102.00 pcf	Density 108.26 pcf	Density 94.00 pcf	Density 102.00 pcf	Density 108.26 pcf
8.69	26.32	42.10	50.00	26.24	42.80	55.56
6.95	20.20	33.52	42.00	20.80	35.20	44.80
4.64	13.96	21.18	28.60	14.60	25.00	30.80
2.32	7.60	11.40	15.80	8.12	14.80	17.52

TABLE 6

$\sigma_{\Delta_1}/\sigma_{\Delta_{max}}$, Ratios Between the Deviator Stress at Which the Stress-Strain Curve Ceases to be Straight Line (σ_{Δ_1}) and the Maximum Deviator Stress ($\sigma_{\Delta_{max}}$)

(Dry Colorado River Sand)

Confining Pressure (σ_3) psi	Rate of Strain 0.625% Per Min.			Rate of Strain 5.000% Per Min.		
	Density 94.00 pcf	Density 102.00 pcf	Density 108.26 pcf	Density 94.00 pcf	Density 102.00 pcf	Density 108.26 pcf
8.69	0.276	0.400	0.427	0.345	0.404	0.540
6.95	0.212	0.358	0.476	0.289	0.369	0.491
4.64	0.000	0.355	0.455	0.000	0.336	0.487
2.32	0.000	0.219	0.254	0.000	0.237	0.286

TABLE 7

Deviator Stress at 0.10 In. Per In. Axial

Strain ($\sigma_{\Delta_{0.1}}$) in PSI

(Dry Colorado River Sand)

Confining Pressure (σ_3) psi	Rate of Strain 0.625% Per Min.			Rate of Strain 5.000% Per Min.		
	Density 94.00 pcf	Density 102.00 pcf	Density 108.26 pcf	Density 94.00 pcf	Density 102.00 pcf	Density 108.26 pcf
8.69	24.90	33.00	34.00	26.10	33.20	35.80
6.95	19.70	27.40	29.40	19.20	28.00	32.00
4.64	13.70	17.10	21.20	14.60	18.40	23.80
2.32	7.60	7.40	10.00	7.65	9.00	10.00

TABLE 8

$\sigma_{\Delta_{0.1}}/\sigma_{\Delta_{max}}$, Ratios Between the Deviator Stress at 0.10 In. Per In. Axial Strain ($\sigma_{\Delta_{0.1}}$) and the Maximum Deviator Stress ($\sigma_{\Delta_{max}}$)

(Dry Colorado River Sand)

Confining Pressure	Rate of Strain 0.625% Per Min.			Rate of Strain 5.000% Per Min.		
(σ_3) psi	Density 94.00 pcf	Density 102.00 pcf	Density 108.26 pcf	Density 94.00 pcf	Density 102.00 pcf	Density 108.26 pcf
8.69	0.946	0.784	0.680	0.995	0.776	0.644
6.95	0.975	0.814	0.700	0.923	0.796	0.714
4.64	0.981	0.807	0.741	1.000	0.736	0.773
2.32	1.000	0.649	0.633	0.942	0.608	0.571

variation of this ratio indicates that as the density, confining pressure, or rate of strain increased, the stress-strain curves peaked more.

The deviator stresses presented above were computed using areas determined from actually measured axial and lateral deformations (as previously explained). The stresses computed using the conventional method of assuming no volume changes during the test were also obtained. When compared, these two stresses were found to have values that are very close. The differences between these values depend on the differences between the areas determined by both methods. It was noticed that the maximum stress in both cases occured at the same axial strain. The assumption of no volume change used in the interpretation of triaxial test results is therefore believed to be justified at low levels of strain. As the axial strain increased the discrepancy between actual and assumed areas became more.

D. Modulus of Deformation

The modulus of deformation E , in the author's opinion, is a soil property that represents axial stress-strain behavior. The modulus of deformation is defined as the secant modulus at any point on the stress-strain curve, and is equal to the ratio of the deviator stress to axial strain at that point. For a perfectly linear stress-strain curve there will be a unique value for E . For a non-linear stress-strain curve, however, E will have various values.

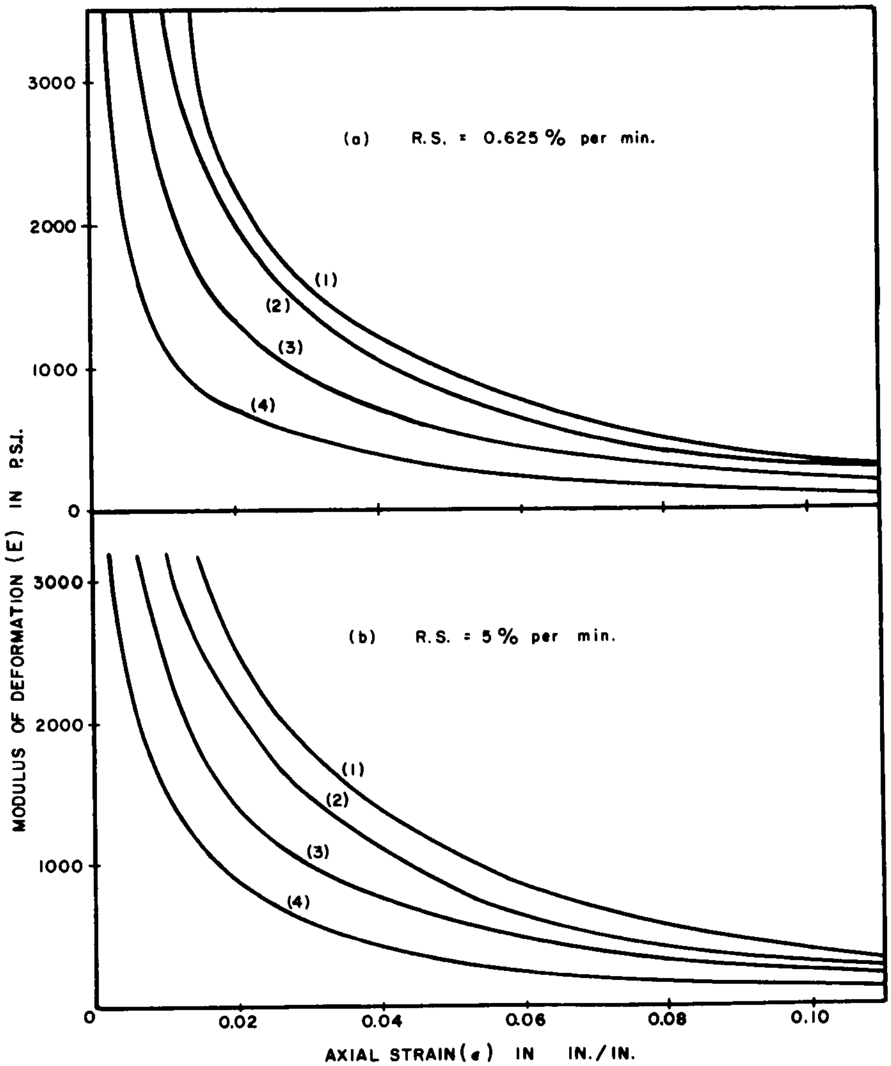
One of the objectives of this laboratory study was to determine the pattern of variation of the modulus of deformation for the various soils studied under one-cyclic static loading, as determined by quick compression triaxial tests. The factors, believed to influence E , that were investigated in this chapter are: the level of strain, the level of stress, the density, the confining pressure, the rate of strain, and the strength of the sand.

The level of strain had a very obvious effect on the value of the modulus of deformation as shown in Figs 5, 6, and 7. In these figures the initial parts of the curves do not appear. The values of the initial tangent modulus will be discussed later, however. Each curve, in any of these figures, represents the average results of two identical triaxial tests. It can be seen that the modulus of deformation for the tested sand, at any rate of strain, density, and confining pressure, decreased as the strain increased. This decrease is very rapid in the lower strain levels and the rate of decrease becomes less at the higher levels of strain.

The effect of the confining pressure σ_3 was also apparent. The modulus of deformation of the sand at any rate of strain, density, and strain level increased as the confining pressure got higher. The density of the sand also had the same effect, that is, the value of the modulus at any rate of strain, confining pressure, and strain level increased with the increase in density. Generally, this increase due to density was more when the density increased from 94 to 102 pcf than it was when the density changed from 102 to 108.26 pcf. This means that the increase in the modulus of deformation with density was more for loose sands than it was for medium to dense sands.

The rate of strain had some effect on the value of the modulus of deformation. For the limited range of rates of strain tested, however, the effect was not large. This effect may be expected to increase at higher strain rates. Generally the value of the modulus, at any density, confining pressure, and level of strain, was higher for the higher rates of strain. The effect of the rate of strain was more pronounced generally at higher densities and at higher confining pressures.

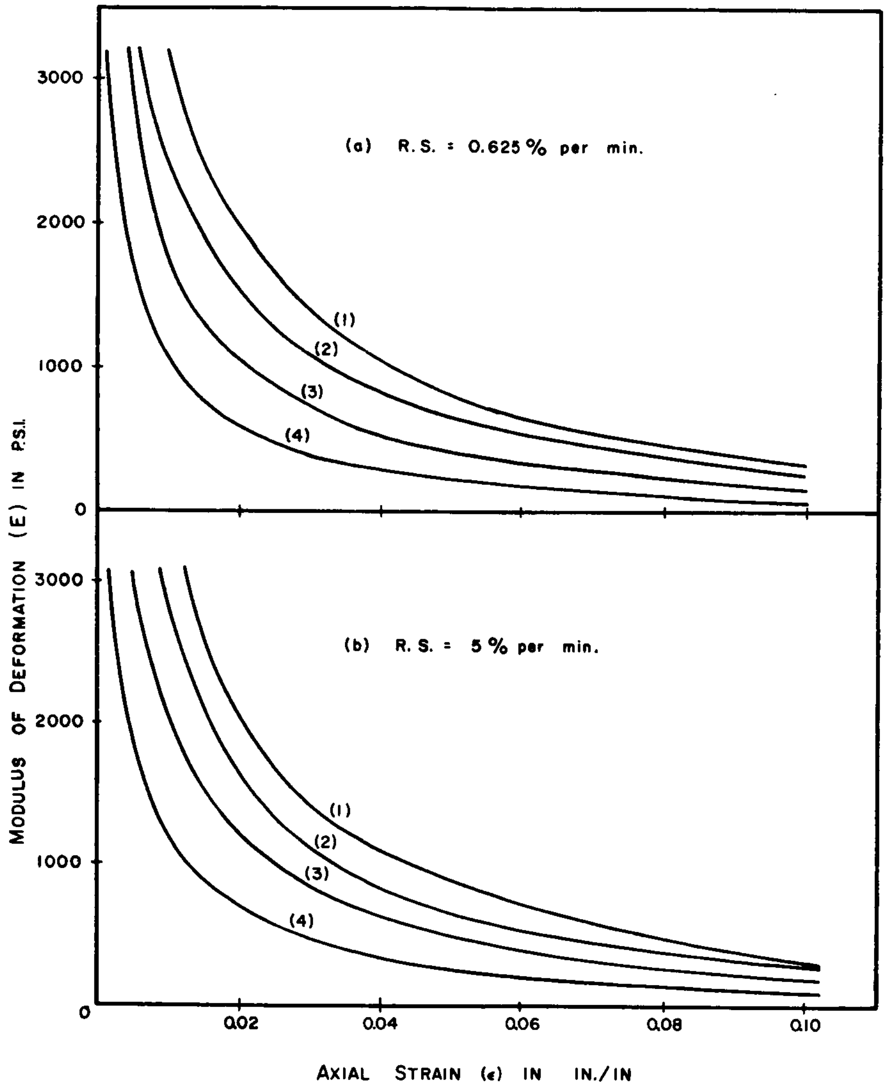
The modulus of deformation of sand, in any one test, starts with a maximum value which is either the slope of the initial linear part of the stress-strain



(1) σ_3 = 8.69 p.s.i.
(2) σ_3 = 6.95 p.s.i.
(3) σ_3 = 4.64 p.s.i.
(4) σ_3 = 2.32 p.s.i.

DENSITY = 108.26 p.c.f.

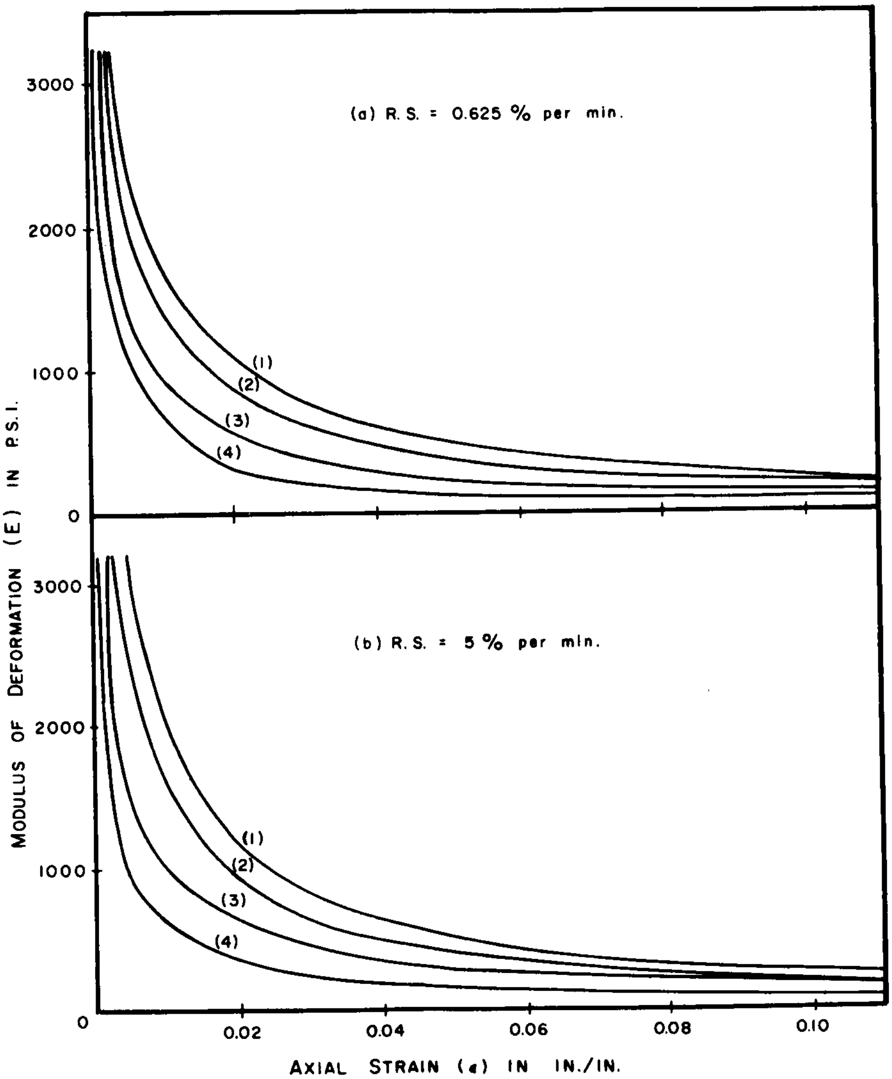
FIG. 5. MODULUS OF DEFORMATION VS. AXIAL STRAIN FOR DRY COLORADO RIVER SAND



(1) σ_3 = 8.69 p.s.i.
(2) σ_3 = 6.95 p.s.i.
(3) σ_3 = 4.64 p.s.i.
(4) σ_3 = 2.32 p.s.i.

DENSITY = 102 p.c.f.

FIG. 6. MODULUS OF DEFORMATION vs. AXIAL STRAIN FOR DRY COLORADO RIVER SAND



(1) σ_3 = 8.69 p.s.i.

(2) σ_3 = 6.95 p.s.i.

(3) σ_3 = 4.64 p.s.i.

(4) σ_3 = 2.32 p.s.i.

DENSITY = 94 p.c.f.

FIG. 7. MODULUS OF DEFOMATION VS. AXIAL STRAIN FOR DRY COLORADO RIVER SAND

curve, or the initial tangent modulus to the curve. If the stress-strain curve is linear at its beginning, the value of the modulus will remain constant for a certain strain, until the linear portion begins to curve. The modulus will then decrease with increased strain at a rate that becomes smaller as the level of strain increases. This rate of decrease is higher for dense sands than it is for loose sands. The reason is that for dense sands the stress-strain curves peak rapidly to a maximum deviator stress and then the stress decreases, with the increase in strain, to a residual value. In case of loose sands, however, the peak of the stress-strain curve is less, occurs at higher strains, is not as well defined in many instances, and the decrease in stress after its maximum value is reached, if any, is much less than for dense sand. The same pattern is also developed, to a lesser extent, for higher and lower confining pressures.

Table 9 gives the values of the initial tangent modulus of deformation E_i for the various densities, confining pressures, and rates of strain investigated. These values represent the slope of the initial linear portion of the stress-strain curve. The values presented show great scatter due to the big effect of the seating error encountered in some tests. Generally, however, one might expect the initial tangent modulus to increase with increased density, confining pressure, or rate of strain. This pattern is not fully developed in Table 9, but would have been if the seating error was completely eliminated.

The modulus of deformation at a point on the stress-strain curve where the deviator stress is equal to half its maximum value will be denoted by E_{50}, and that at the maximum stress E_{100}. Tables 10 and 11 give the values of these moduli for the various conditions studied. Again some scatter in the shown values is apparent but the general trend is obvious. These values generally increase as the density, confining pressure, and rate of strain increase. The results presented also show that for the investigated range, the effect of the rate of strain was not conclusive.

The values presented in Tables 9, 10 and 11 point out that the level of stress has definitely a tremendous effect on the modulus of deformation of the sand. This fact is shown in Fig 8 where the modulus of deformation is drawn versus the factor of safety* for various densities confining pressures, and rates of strain. The factor of safety in the present study is defined as the ratio of the maximum deviator stress to that at which the modulus is computed. For example, E_{100} is drawn at a factor of safety equal to one and E_{50} at two and so on. The factor of safety as defined above represents the level of stress. It can be seen from this figure that the modulus of deformation increased as the factor of safety increased, that is as the level of stress was reduced, till it reaches a maximum value (the initial modulus.) The rate of increase, however, was higher at the lower factors of safety and decreased as the factor of safety increased.

Figures 9 and 10 show the effect of the level of stress in the form of non-dimensional curves. In these figures there is some scatter in results due to the various factors, discussed before, that may effect the stress-strain properties in general, and also because we are using various rates of strain in drawing these points (due to the little effect of the rates of strain used). The modulus of deformation E in this case is divided by the confining pressure $\sigma_3 \tan \phi$ in Fig 10 to give the required non-dimensional curves.

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*The term "factor of safety" as used here has different significance than ordinari y used in engineering practice. In this investigation the term is used as an indication of the level of stress at a point on the stress-strain curve with respect to the maximum stress.

TABLE 9

Initial Modulus of Deformation (E_i) in PSI x 10^2

(Dry Colorado River Sand)

Confining Pressure (σ_3) psi	Rate of Strain 0.625% Per Min.			Rate of Strain 5.000% Per Min.		
	Density 94.00 pcf	Density 102.00 pcf	Density 108.26 pcf	Density 94.00 pcf	Density 102.00 pcf	Density 108.26 pcf
8.69	35.15	65.00	96.00	87.62	100.00	86.00
6.95	58.00	42.39	60.00	70.00	84.00	73.00
4.64	50.00	39.44	46.00	52.00	59.66	37.50
2.32	33.60	36.00	40.00	34.00	38.00	53.00

TABLE 10

Modulus of Deformation at Half the Maximum Deviator

Stress (E_{50}) in PSI x 10^2

(Dry Colorado River Sand)

Confining Pressure	Rate of Strain 0.625% Per Min.			Rate of Strain 5.000% Per Min.		
(σ_3) psi	Density 94.00 pcf	Density 102.00 pcf	Density 108.26 pcf	Density 94.00 pcf	Density 102.00 pcf	Density 108.26 pcf
8.69	28.00	50.12	78.13	52.48	92.27	66.95
6.95	25.25	37.24	58.33	31.52	55.00	56.00
4.64	17.45	42.36	40.86	15.20	41.67	36.67
2.32	19.00	27.14	22.57	10.15	23.13	31.29

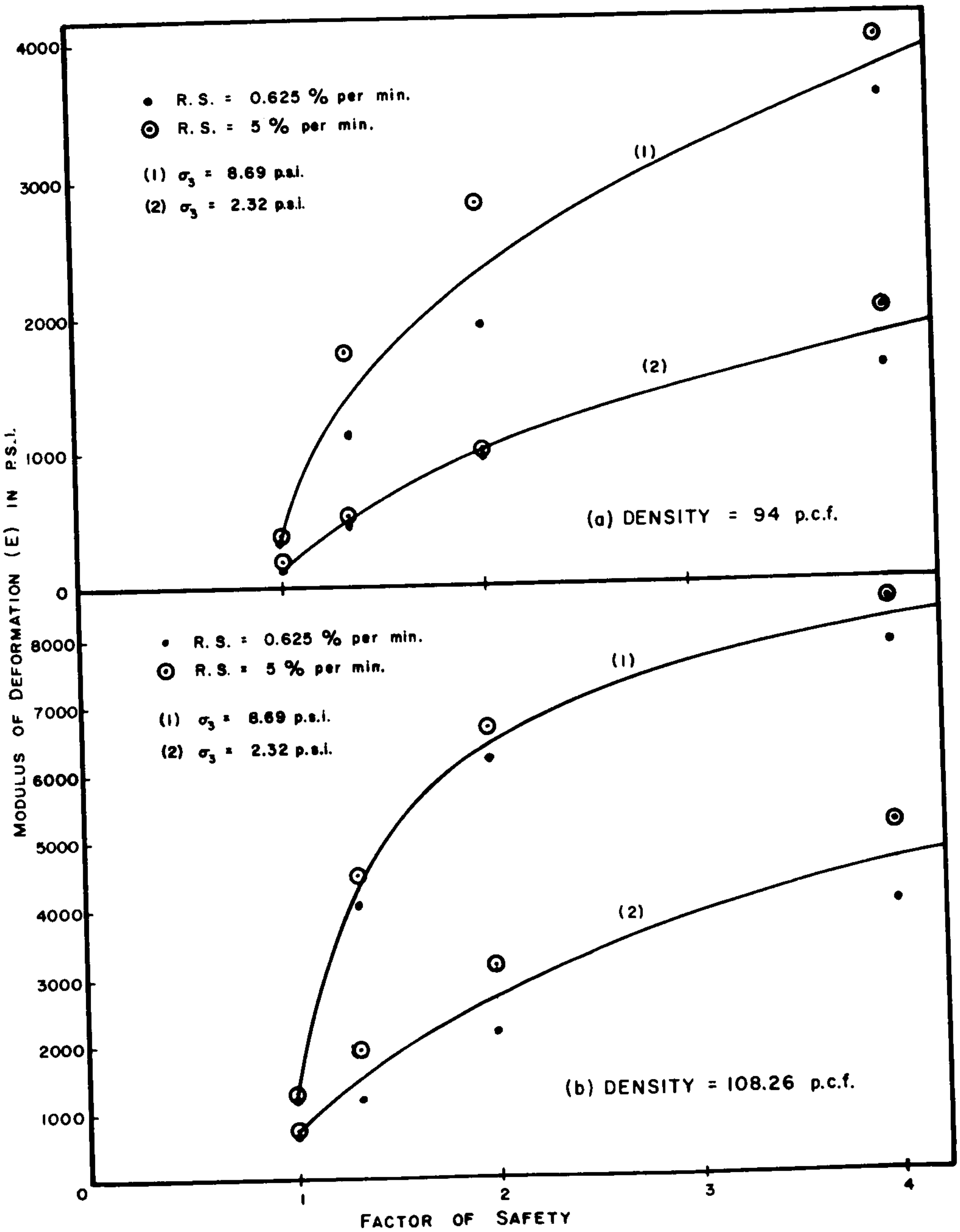
TABLE 11

Modulus of Deformation at Maximum Deviator

Stress (E_{100}) in PSI x 10^2

(Dry Colorado River Sand)

Confining Pressure (σ_3) psi	Rate of Strain 0.625% Per Min.			Rate of Strain 5.000% Per Min.		
	Density 94.00 pcf	Density 102.00 pcf	Density 108.26 pcf	Density 94.00 pcf	Density 102.00 pcf	Density 108.26 pcf
8.69	3.41	11.23	11.63	2.73	11.89	13.43
6.95	2.15	8.38	13.13	3.35	13.54	11.49
4.64	1.59	6.83	8.41	1.46	7.35	5.88
2.32	0.76	5.70	4.51	0.94	3.45	6.49



DRY COLORADO RIVER SAND

FIG. 8. EFFECT OF THE LEVEL OF STRESS ON THE MODULUS OF DEFORMATION

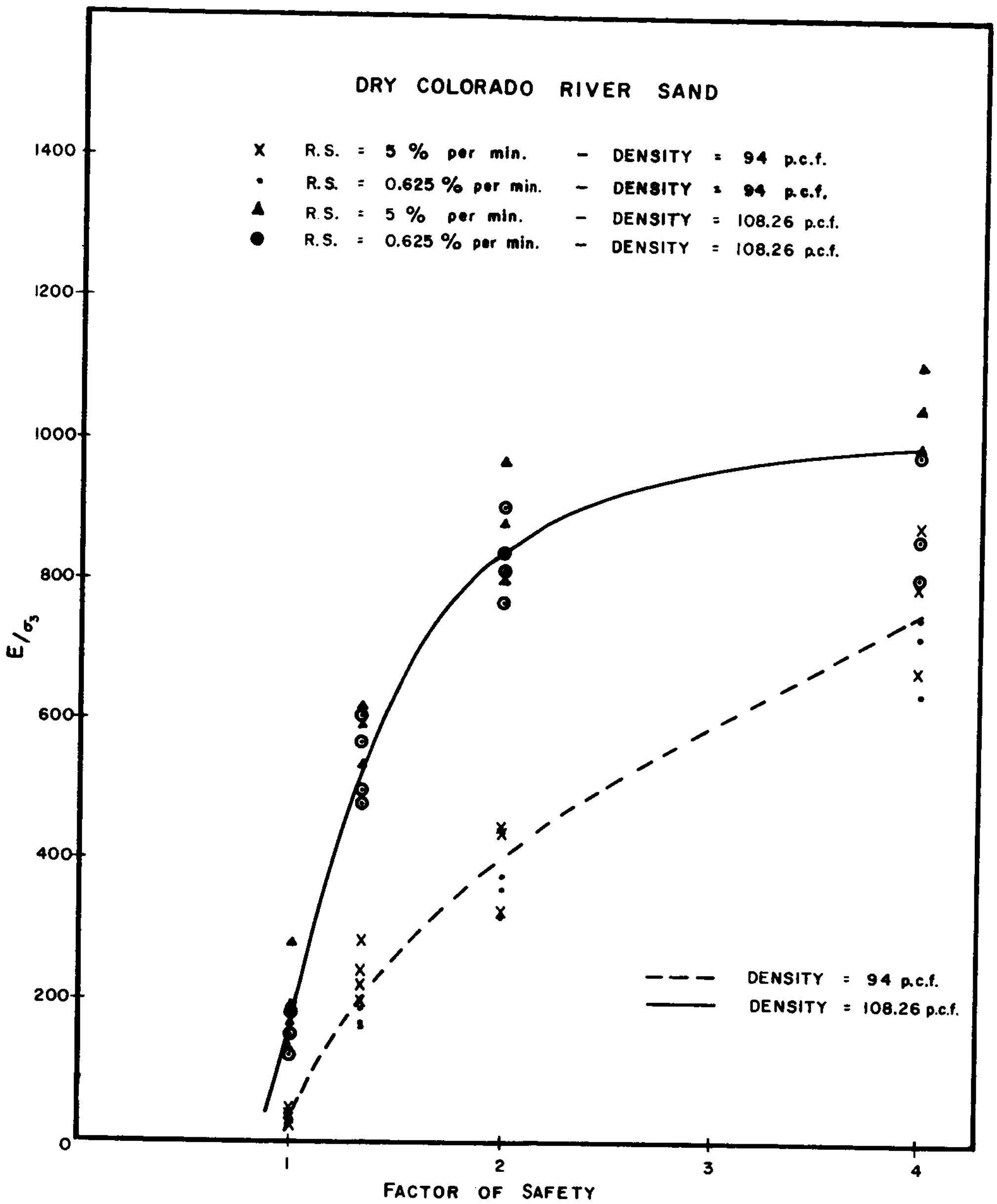
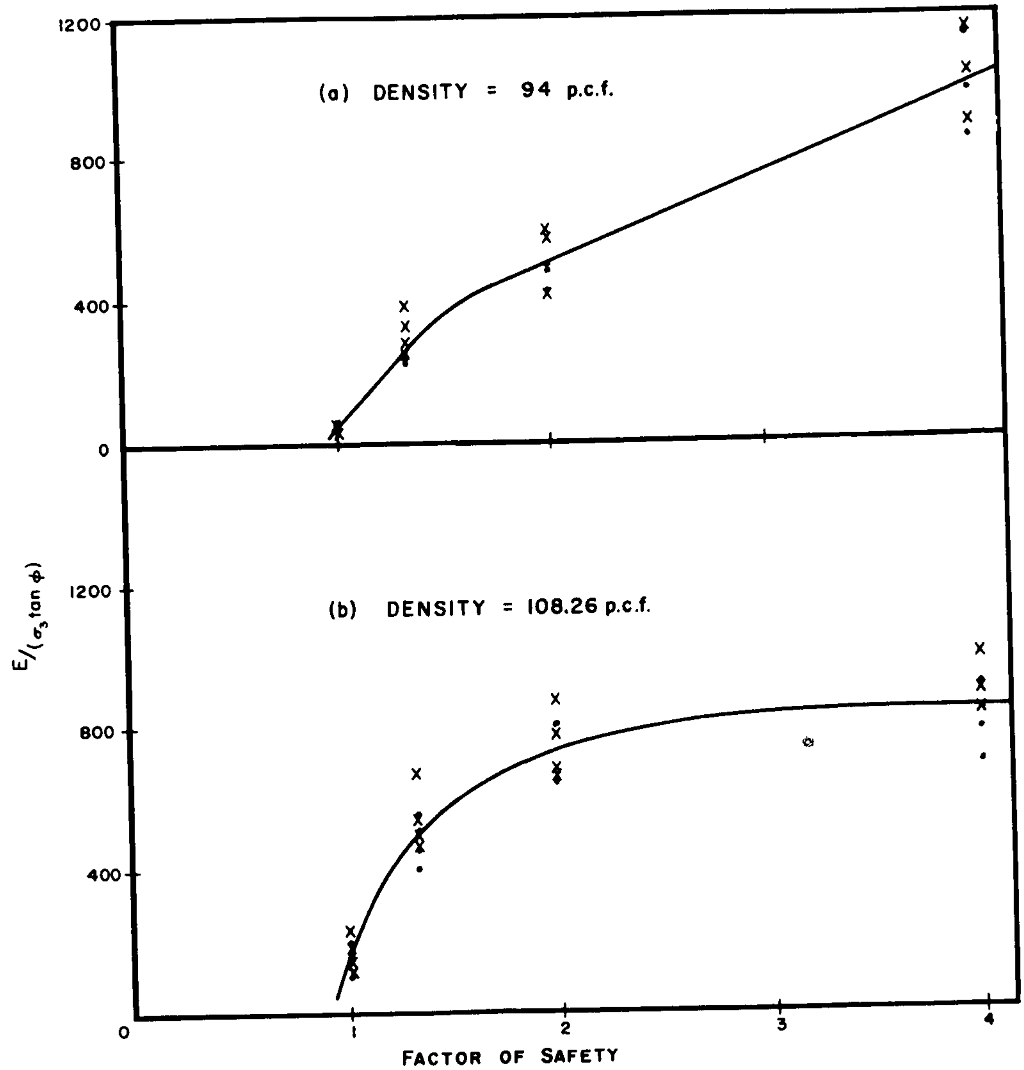


FIG. 9. EFFECT OF THE LEVEL OF STRESS ON THE MODULUS OF DEFORMATION (NON – DIMENSIONAL CURVES)



DRY COLORADO RIVER SAND

• R. S. = 0.625 % per min.

X R. S. = 5 % per min.

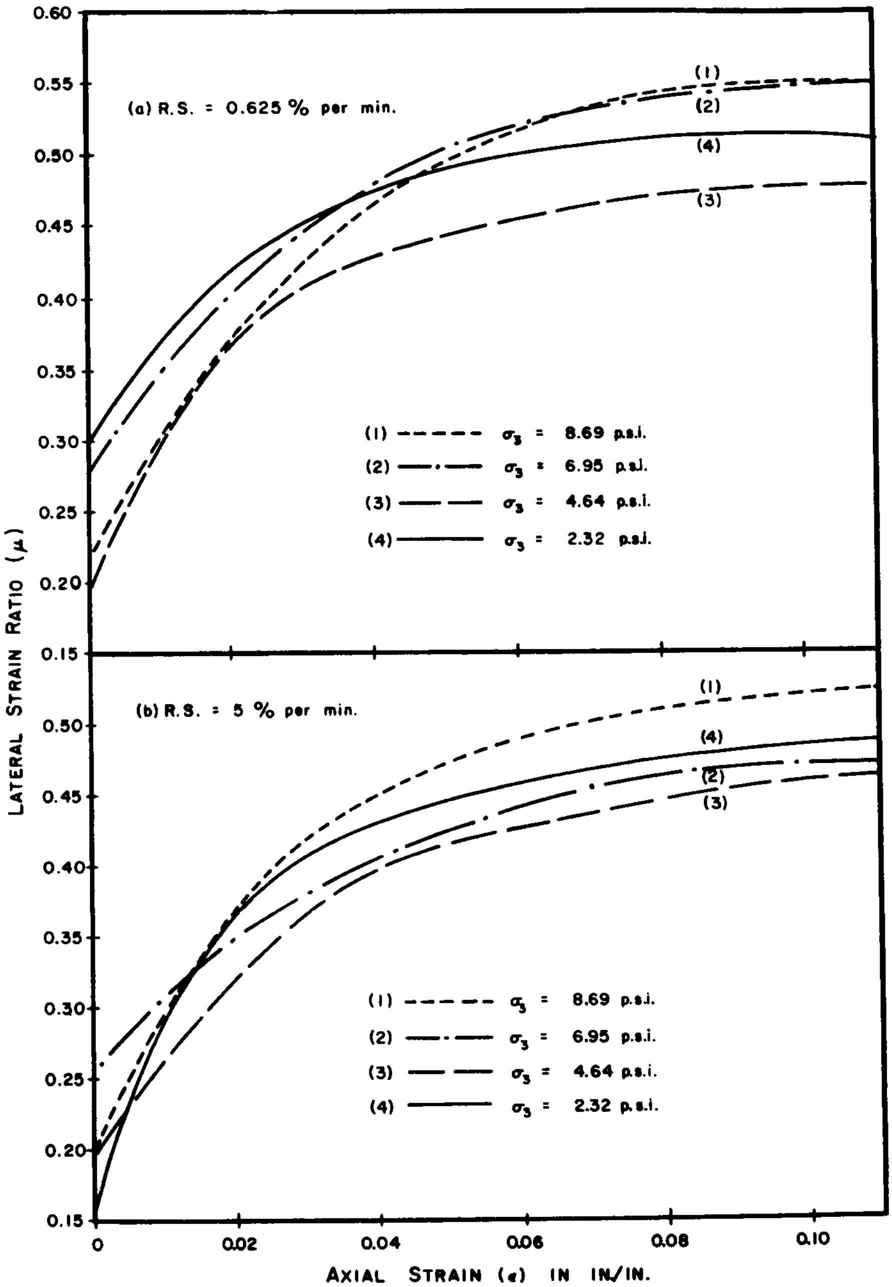
FIG. 10. EFFECT OF THE LEVEL OF STRESS ON THE MODULUS OF DEFORMATION
(NON-DIMENSIONAL CURVES)

E. Lateral Strain Ratio

The author believes that the lateral deformation behavior of soils can be well represented by what was termed the lateral strain ratio μ . The lateral strain ratio is defined to be the ratio of the lateral strain to the axial strain of the triaxial test specimen at any time during the test. The lateral strain is obtained by dividing the lateral deformation, of the cylindrical test specimen, by the original diameter of the sample, while the axial strain is taken to be the axial deformation divided by the original height. As previously discussed (in Art. 4.6-A) the average axial deformation is measured directly by the axial dial extensometer, but the average lateral deformation is taken to be half the value at the mid-height of the specimen (which is equal to the mean value of the four readings of the lateral deformation extensometers mounted at the middle of the sample.)

It should be pointed out here that for an elastic material the lateral strain ratio is nothing but Poisson's ratio which in this case has a single unique value. This is not the case, however, for soils and other non-elastic materials.

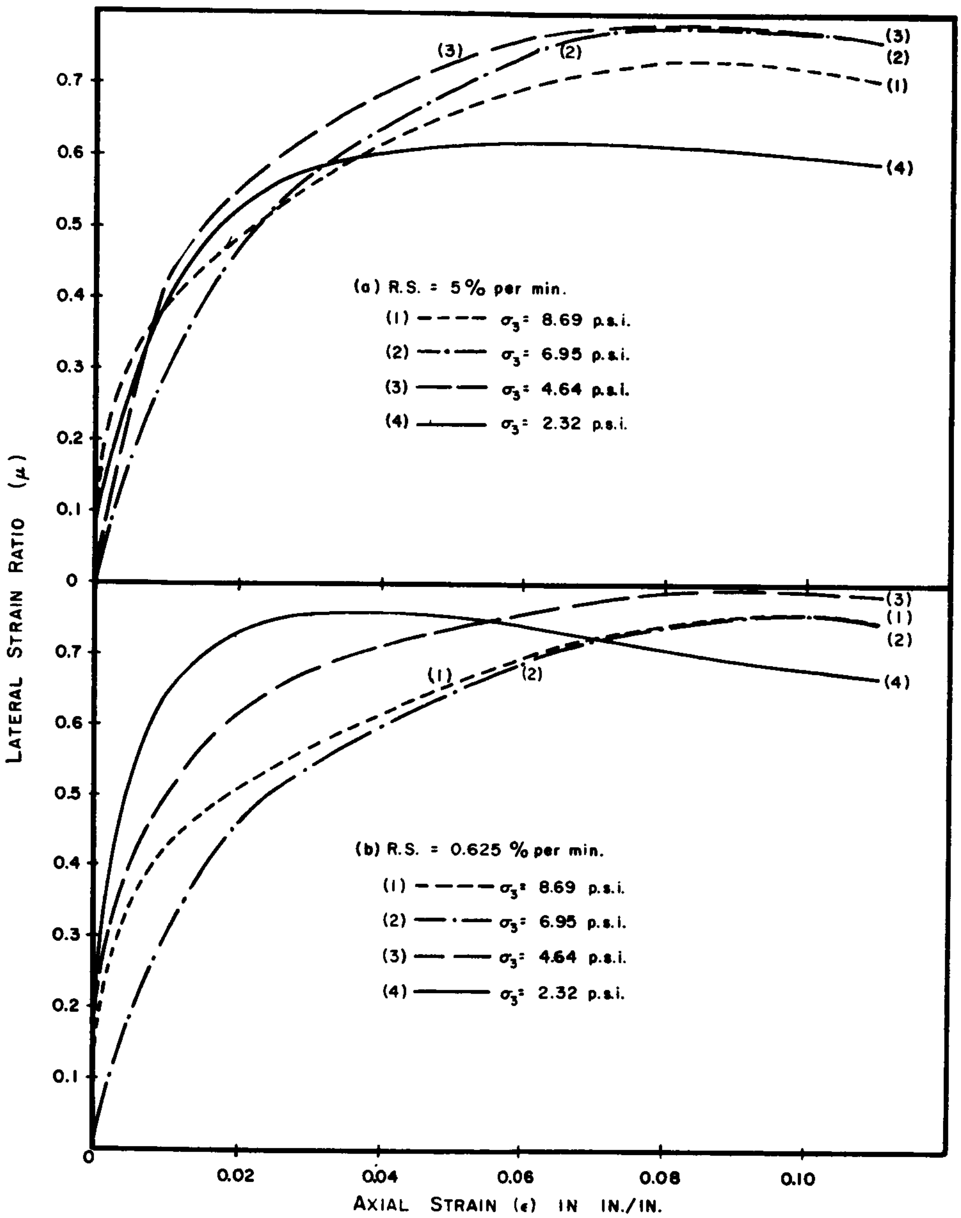
Figures 11, 12, and 13 show the effect of the level of strain, confining pressure, density, and rate of strain on the lateral strain ratio. It is obvious that the level of strain has a tremendous effect on the value of μ . As the strain (at any one density, confining pressure, and rate of strain) increased, the lateral strain ratio also increased. This increase, however, was more in dense than in loose sand. With the higher densities the value of μ decreased slightly, in some tests, at higher strain levels and after failure had started. It should be pointed out, however, that this reduction occurred only after failure had begun and the deformation of the specimen became excessive



DRY COLORADO RIVER SAND

DENSITY = 94 p.c.f.

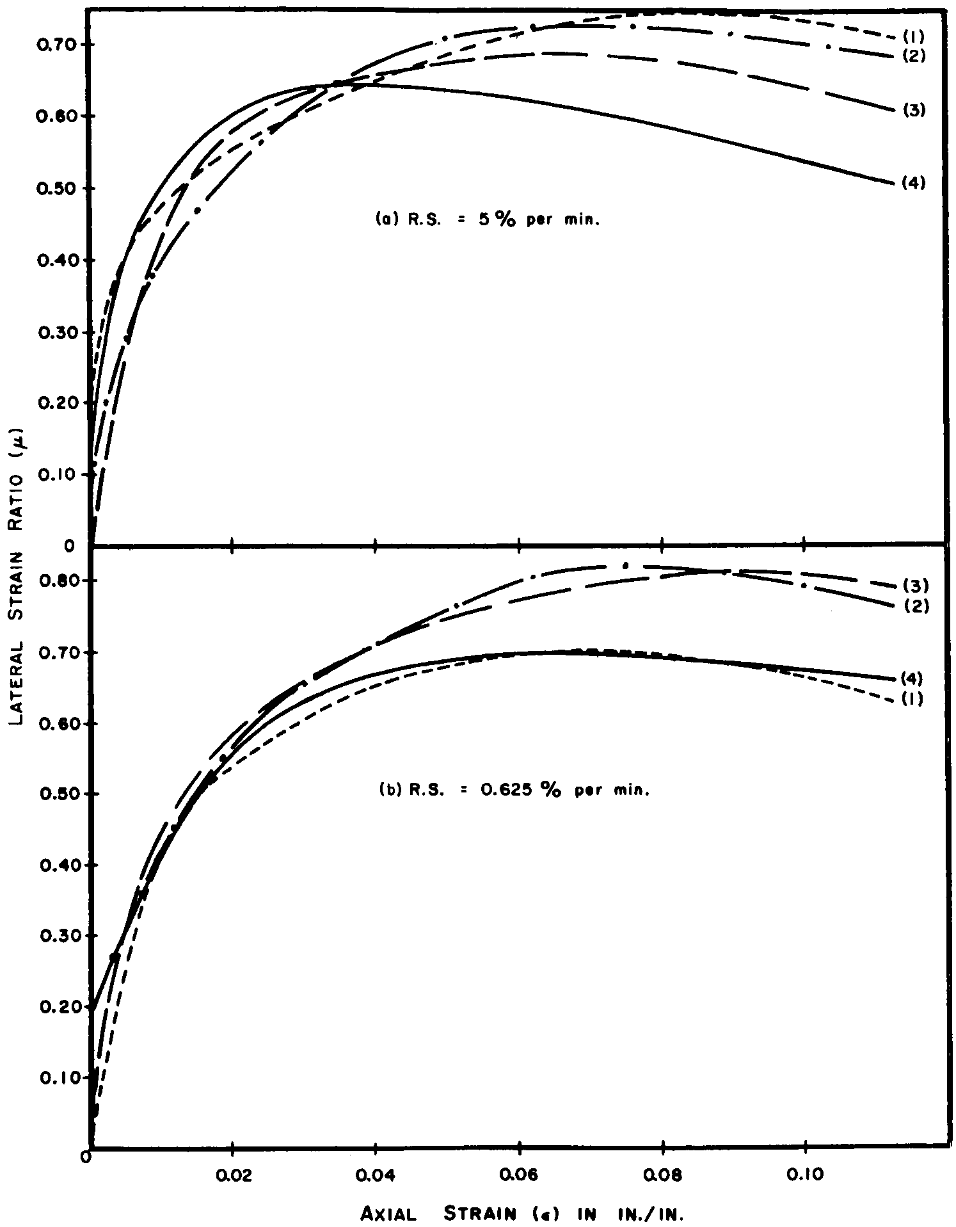
FIG. 11. LATERAL STRAIN RATIO VS. AXIAL STRAIN



DRY COLORADO RIVER SAND

DENSITY = 102 p.c.f.

FIG. 12. LATERAL STRAIN RATIO VS. AXIAL STRAIN



(1) σ_3 = 8.69 p.s.i.　(3) σ_3 = 4.64 p.s.i.

(2) σ_3 = 6.95 p.s.i.　(4) σ_3 = 2.32 p.s.i.

DENSITY = 108.26 p.c.f.

FIG. 13. LATERAL STRAIN RATIOS FOR DRY COLORADO RIVER SAND

due to bulging, tilting, and movement along the failure plane. Thus less weight should be put on the value of μ at these higher strain levels where failure has already progressed to a far point.

Figures 11, 12, and 13 also emphasize the effect of sand density on the lateral strain ratio. The value of μ at any strain level, confining pressure and rate of strain, increased at higher densities. The only exceptions to this rule occurred at very small strains where the lateral strain ratios for loose density conditions were sometimes higher than those for dense sands. The effect of density is also demonstrated in Tables 12 and 13. From these tables it can be seen that, generally, the value of μ , at any stress level, confining pressure, and rate of strain, increased with the increase in density.

Tables 12 and 13 also show the effect of the level of stress on the lateral strain ratio. These tables reveal the fact that μ increased with the decrease in the factor of safety (representing an increase in the stress level).

Although the presented results point out the effect of the confining pressure on the value of μ , no regular pattern could be developed in this regard, for the limited range of vacuums investigated. The only conclusion that might be reached with respect to the effect of the confining pressure is an average curve that would represent the value of μ in the entire range of pressures studied. Some of the factors that could have contributed to the irregular pattern of results obtained in this case were already given in Art. 4.6-C. In addition, the assumptions made in calculating the average lateral deformation may not have been 100% correct, and the very limited range of confining pressures investigated in the present study was not enough to show big differences in results. It is expected that if a bigger range of pressures is applied, the influence on the lateral strain ratio will be regular and more obvious.

TABLE 12

Lateral Strain Ratio at Maximum

Deviator Stress (μ_{100})

(Dry Colorado River Sand)

Confining Pressure (σ_3) psi	Rate of Strain 0.625% Per Min.			Rate of Strain 5.000% Per Min.		
	Density 94.00 pcf	Density 102.00 pcf	Density 108.26 pcf	Density 94.00 pcf	Density 102.00 pcf	Density 108.26 pcf
8.69	0.545	0.600	0.660	0.515	0.590	0.660
6.95	0.548	0.610	0.675	0.440	0.550	0.700
4.64	0.475	0.680	0.685	0.460	0.655	0.640
2.32	0.480	0.730	0.655	0.290	0.575	0.635

TABLE 13

Lateral Strain Ratio at Half the Maximum

Deviator Stress (μ_{50})

(Dry Colorado River Sand)

Confining Pressure (σ_3) psi	Rate of Strain 0.625% Per Min.			Rate of Strain 5.000% Per Min.		
	Density 94.00 pcf	Density 102.00 pcf	Density 108.26 pcf	Density 94.00 pcf	Density 102.00 pcf	Density 108.26 pcf
8.69	0.268	0.310	0.300	0.250	0.300	0.390
6.95	0.310	0.170	0.290	0.270	0.110	0.260
4.64	0.245	0.340	0.250	0.238	0.160	0.230
2.32	0.325	0.350	0.280	0.210	0.220	0.320

The influence of the limited range of rates of strain used in running the various tests was not great. Generally, the lateral strain ratio (at any density, confining pressure, and at any one level of stress or strain) decreased as the rate of strain increased. This variation was not always regular, however. Higher rates of strain must first be studied before any pattern of variation could finally be reached.

F. Volume Changes

Changes in volume that take place within a soil sample during shear are generally measured for saturated soils and in drained triaxial tests where the actual volume of the water expelled from the specimen is measured in a burette. In the present investigation, volume changes of dry Colorado River sand specimens were computed using the measured axial and lateral deformations of the triaxial specimen.

Knowing the original dimensions of the specimen and the average lateral and axial deformation of the sample at any stage during the test, the volume change at this point can be computed in the following manner:

$A_o = \pi D_o^2/4$

$V_o = A_o H_o$

where

D_o = original diameter of the test specimen.

A_o = original area of the test specimen.

H_o = original height of the test specimen.

V_o = original volume of the test specimen.

$H = H_o - \Delta$

$D = D_o + \Delta_1$

$A = \pi D^2/4$

$$V = A\,H$$

$$\Delta V = V - V_o$$

where

H = height of the specimen at any time during the test.

D = diameter of the specimen at any time during the test.

A = area of the specimen at any time during the test.

V = volume of the specimen at any time during the test.

Δ = average axial deformation at any time during the test.

Δ_1 = average lateral deformation at any time during the test.

ΔV = volume change at any time during the test.

The volume changes obtained in this way for the various tests showed that in the case of loose sand (density of 94 pcf) there was a volume decrease in the sample during the entire test, that is ΔV was negative. Generally it was observed that the amount of volume decrease increased as the strain increased during the test. The effect of confining pressures was obvious but not regular in all cases, because of the small differences in the amount of vacuums used together with the reasons given before in Art. 4.6-C. The rate of strain had some effect on the volume decrease of the sample but again no regular pattern could be detected due to the limited range studied. A wider range of confining pressures and rates of strain should be investigated before any final conclusions could be reached in this regard. Fig 14 shows examples of such variation.

In the case of dense sand (108.26 pcf density) the volume changes followed a totally different pattern. In each test there was a small amount of volume decrease at the beginning (at low strains), then the sample started expanding (see Fig 14). The volume increase became more as the test progressed until failure began. The reliability of the lateral deformation measurements after

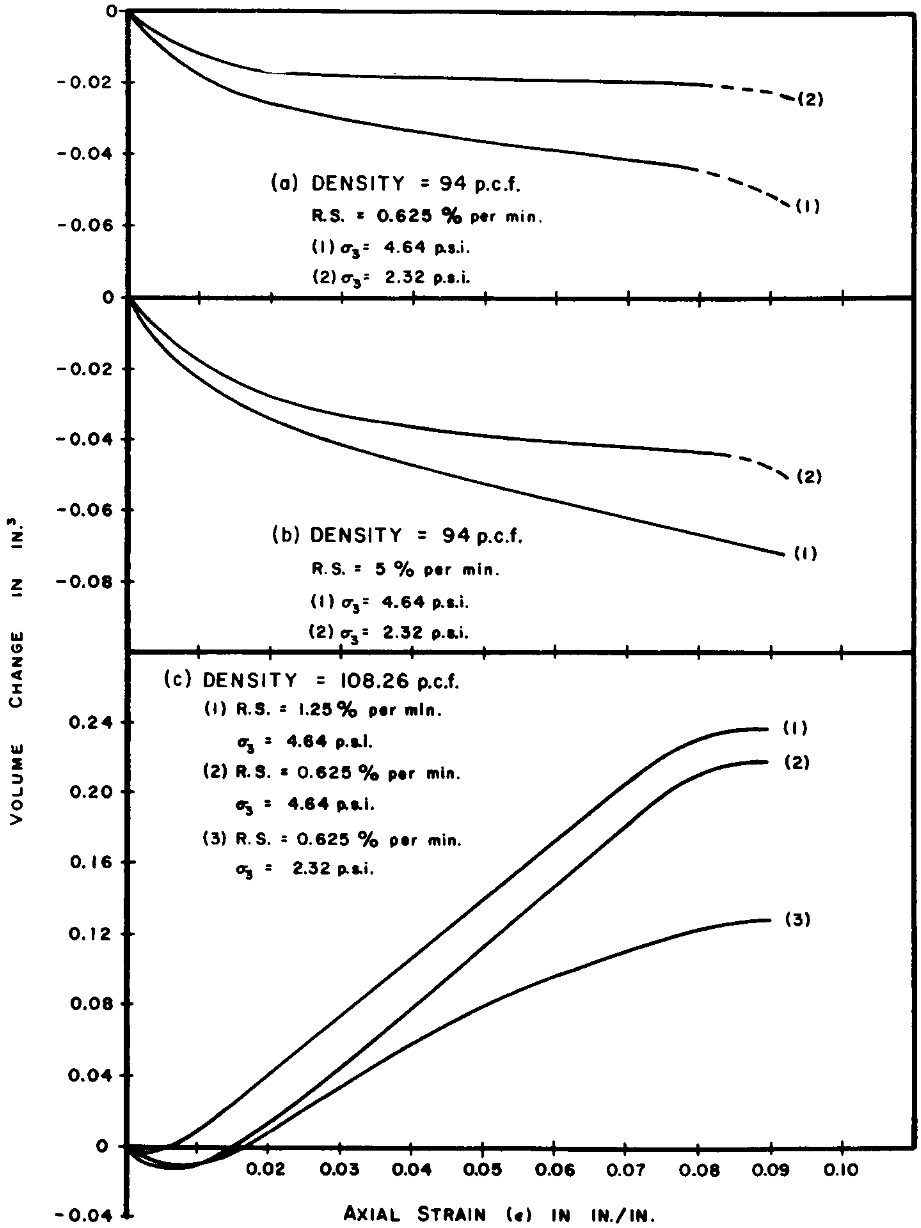


FIG. 14. VOLUME CHANGES IN DRY SPECIMENS OF COLORADO RIVER SAND

this stage of the test is questioned because of the excessive amount of deformation, deformation, bulging, tilting, and sliding of the sample along the failure plane. This is the reason why volume changes at the higher levels of strain are not considered, although it must be pointed out here that in some tests the volume increase started to drop down at these higher strains and after failure was reached. The effect of confining pressure and rate of strain on the volume changes of the test specimens of dense sand were apparent, but not consistent in all tests for reasons given earlier. The validity of this statement has yet to be proved for higher pressures and strain rates.

Comparing the amount of volume change in loose and dense sand, it was found that the volume decrease of the former was about equal to the volume increase in the latter, at small strains. The value of volume increase of dense sand, however, was much more than the volume decrease in loose sand, at larger strains.

Samples with medium density, of 102 pcf, behaved in the same manner as those of dense sand. The amount of volume changes were less, however.

The real conclusion that could be reached from this limited investigation of volume changes in dry sand specimens during shear was the development of a new way of measuring volume changes for dry sand and the establishment of the pattern of such changes for various densities. More tests are required, however, to fully investigate the effect of confining pressure and rate of strain on volume changes.

It should be pointed out that the volume changes measured in this study represent average values for the entire specimen. The volume changes taking place at the ends of the specimen are different from those at the failure plane due to end restraint by friction. Volume changes occur in sand specimens during shear due to movement of the particles. The sand particles move to a more dense structure when loose sand is sheared, thus reducing the volume of voids and causing a volume decrease. In case of dense sand, the particles during shear move to a less compact structure producing a volume increase.

CHAPTER V

TRIAXIAL COMPRESSION TESTS ON DRY SAND

5.1 General

In this chapter a series of tests is reported where the conventional form of triaxial test was used. The confining pressure is applied in the form of air pressure. This form of test was chosen to make possible the application of high pressures.

Tests were run on dry Ottawa sand at about its maximum and minimum densities. The confining pressures were rather high, representing great depths. All tests were performed using the same rate of loading. It was decided not to measure the lateral deformation of the test specimen in these tests because such measurement is too complicated and time-consuming.

The main objective of these tests was the study of the axial stress-deformation characteristics of another sand under various conditions of density and high confining pressures, and to determine whether or not there is any variation from the patterns developed in the previous chapter.

5.2 Soil Used and Sample Preparation

The present series of tests was run on an air-dry, clean sand known as Ottawa sand. This is a fairly uniform fine sand that has a yellowish-white color. The grain size distribution curve is given in Fig 3. The sand particles are generally spherical in shape and have a rather smooth texture. The hygroscopic moisture content of the sand is 0.15% and its specific gravity 2.68. The sand was prepared in the same manner as the Colorado River sand, except that all the particles in this case passed the number 40 U.S. Standard sieve.

Sand specimens were prepared at two different densities that represented the maximum and minimum values for this sand. These two densities were 98 and 109.6 pcf. The test specimens were prepared according to the procedure outlined in Art. 4.4. The vacuum applied to the specimen through its base at the end of sample preparation was about 9.0 psi, which was always less than the confining pressure utilized in these tests.

5.3 Test Equipment and Procedure

The test equipment and set-up was similar to the one described in Art. 4.3 with only one exception. In this case the triaxial cell had the lucite cylinder on. The cylinder represented the confining pressure chamber. This pressure was applied in the form of air pressure through an opening at the top of the cell (see Fig 4) and was regulated by a pressure regulator. The air pressure was measured by a pressure gage reading up to 60 psi and having a sensitivity of 1.0 psi. Both the pressure regulator and gage were placed between the pressure source and the triaxial cell (and both were manufactured by HOKE, Inc., USA).

After the sample was prepared the triaxial cell was put together, with the lucite cylinder in place. The air pressure was then applied slowly in the triaxial chamber and the vacuum at the base of the specimen reduced simultaneously at the same rate until all the vacuum was removed and an equal amount of pressure applied. The air pressure was then allowed to build up gradually using the regulator, until the confining pressure desired was reached. The valve at the bottom of the specimen was then closed, the loading rod lowered, and the test continued as outlined in Art. 4.5 (except that no lateral deformation readings were taken).

Specimens at maximum and minimum densities were tested under 20 and 40 psi confining pressures. In all tests the load was applied at a speed of 0.064 in. per minute. This speed corresponds to a rate of strain equal to 2.0 per cent per minute (sample height being 3.2 in.). Each test was repeated twice to insure accuracy and repeatability of the data. The results of the two tests (representing one case) were then averaged graphically, and the average considered in the analysis.

5.4 Results and Discussion

The results of triaxial tests expressing the axial stress-deformation characteristics of dry Ottawa sand are presented in the following pages. The pattern of variation of the stresses, strains, and modulus of deformation of Ottawa sand with the variables investigated is the same as that for the Colorado River sand. This pattern was better developed though and the results were more consistent and less scattered, in the case of Ottawa sand. The reason for this observation is believed to be the wide range of confining pressures used.

The stress-strain curves for the investigated sand are shown in Appendix A. The Mohr circles and envelopes, and the angle of internal friction are given in Appendix B (ϕ increased as the density increased).

A. Stresses and Strains

A summary of all results on Ottawa sand is presented in Tables 14 and 15. It can be seen that as the density or confining pressure increases the strain ϵ_i increased, ϵ_{100} decreased, and the ratio of the two strains $\epsilon_i/\epsilon_{100}$ decreased. Also there was an increase in the value of the deviator stress, at any level of stress or strain, with the increase in density and

TABLE 14

STRAINS AND MODULUS OF DEFORMATION

FOR DRY OTTAWA SAND

(a) ϵ_1 in In./In.

Confining Pressure (psi)	Density (pcf) 98.0	Density (pcf) 109.6
20	0.0062	0.0068
40	0.0072	0.0081

(b) ϵ_{100} in In./In.

Confining Pressure (psi)	Density (pcf) 98.0	Density (pcf) 109.6
20	0.110	0.044
40	0.106	0.052

(c) $\epsilon_1 / \epsilon_{100}$

Confining Pressure (psi)	Density (pcf) 98.0	Density (pcf) 109.6
20	0.0564	0.1546
40	0.0834	0.1558

(d) E_1 in PSI

Confining Pressure (psi)	Density (pcf) 98.0	Density (pcf) 109.6
20	3709.67	7647.05
40	7083.33	12345.60

(e) E_{50} in PSI

Confining Pressure (psi)	Density (pcf) 98.0	Density (pcf) 109.6
20	3709.67	7647.05
40	7083.33	12345.60

(f) E_{100} in PSI

Confining Pressure (psi)	Density (pcf) 98.0	Density (pcf) 109.6
20	402.70	1783.18
40	907.55	2765.00

TABLE 15

AXIAL STRESSES FOR DRY OTTAWA

SAND

(a) σ_{Δ_1} in PSI

Confining Pressure (psi)	Density (pcf) 98.0	Density (pcf) 109.6
20	23.00	52.00
40	51.00	100.00

(b) $\sigma_{\Delta_{max}}$ in PSI

Confining Pressure (psi)	Density (pcf) 98.0	Density (pcf) 109.6
20	44.30	78.46
40	96.20	143.80

(c) $\sigma_{\Delta_{0.2}}$ in PSI

Confining Pressure (psi)	Density (pcf) 98.0	Density (pcf) 109.6
20	41.50	52.00
40	86.70	94.20

(d) $\sigma_{\Delta_1}/\sigma_{\Delta_{max}}$

Confining Pressure (psi)	Density (pcf) 98.0	Density (pcf) 109.6
20	0.519	0.663
40	0.530	0.695

(e) $\sigma_{\Delta_{0.2}}/\sigma_{\Delta_{max}}$

Confining Pressure (psi)	Density (pcf) 98.0	Density (pcf) 109.6
20	0.937	0.663
40	0.901	0.655

and confining pressure. The ratio $\sigma_{\Delta_1}/\sigma_{\Delta_{max}}$ increases, while $\sigma_{\Delta_{0.2}}/\sigma_{\Delta_{max}}$ decreased, with increasing density and confining pressure. It must be pointed out here that the stress-strain curves remained linear to a point beyond half the maximum deviator stress in almost all tests. Also that this initial straight line portion is greater than in the case of Colorado River sand, mainly because of the high confining pressures used. The strains, at any level of stress, were generally a bit larger in this case than in the previous chapter, in spite of the high pressures, because of the smaller size of the sand particles and their shape and texture, and also for the more uniform grading of Ottawa sand.

B. Modulus of Deformation

The initial tangent modulus of deformation of dry Ottawa sand is given in Table 14(d). Its value increased with the increase in density and confining pressure. Table 14(e) gives E_{50} and Table 14(f) E_{100}. The modulus of deformation of the sand, at any level of stress or strain, increased as the density or pressure was raised (as shown in Figs 15 and 16). These two figures also indicate that the modulus decreased with the increase in the level of strain or stress. Fig 17 gives the modulus of deformation in a non-dimensional form.

From the results of triaxial tests on the two sands it can be concluded that the stress-strain behavior of sands in general can be expected to follow the same general patterns described before.

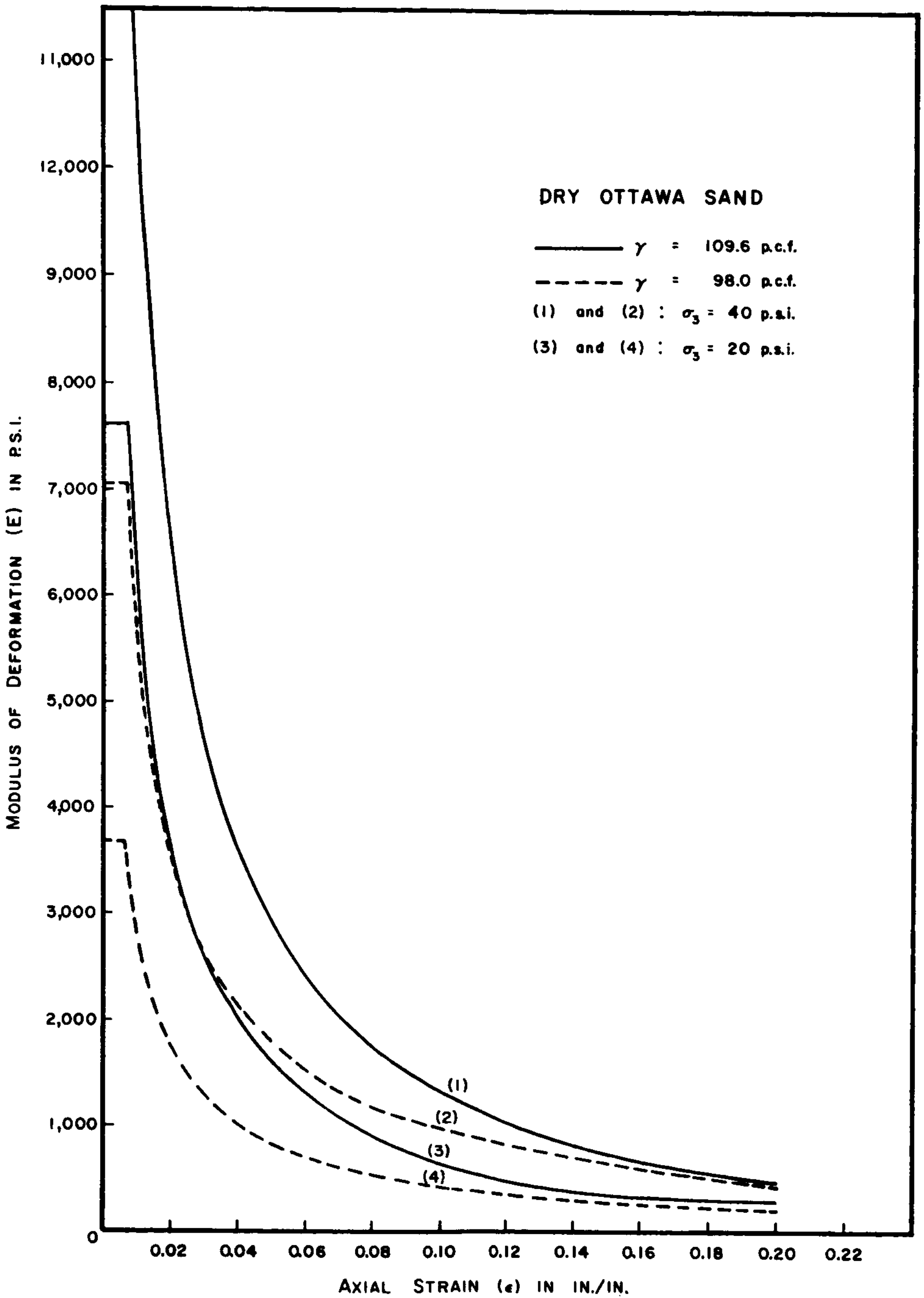


FIG. 15. MODULUS OF DEFORMATION VS. AXIAL STRAIN

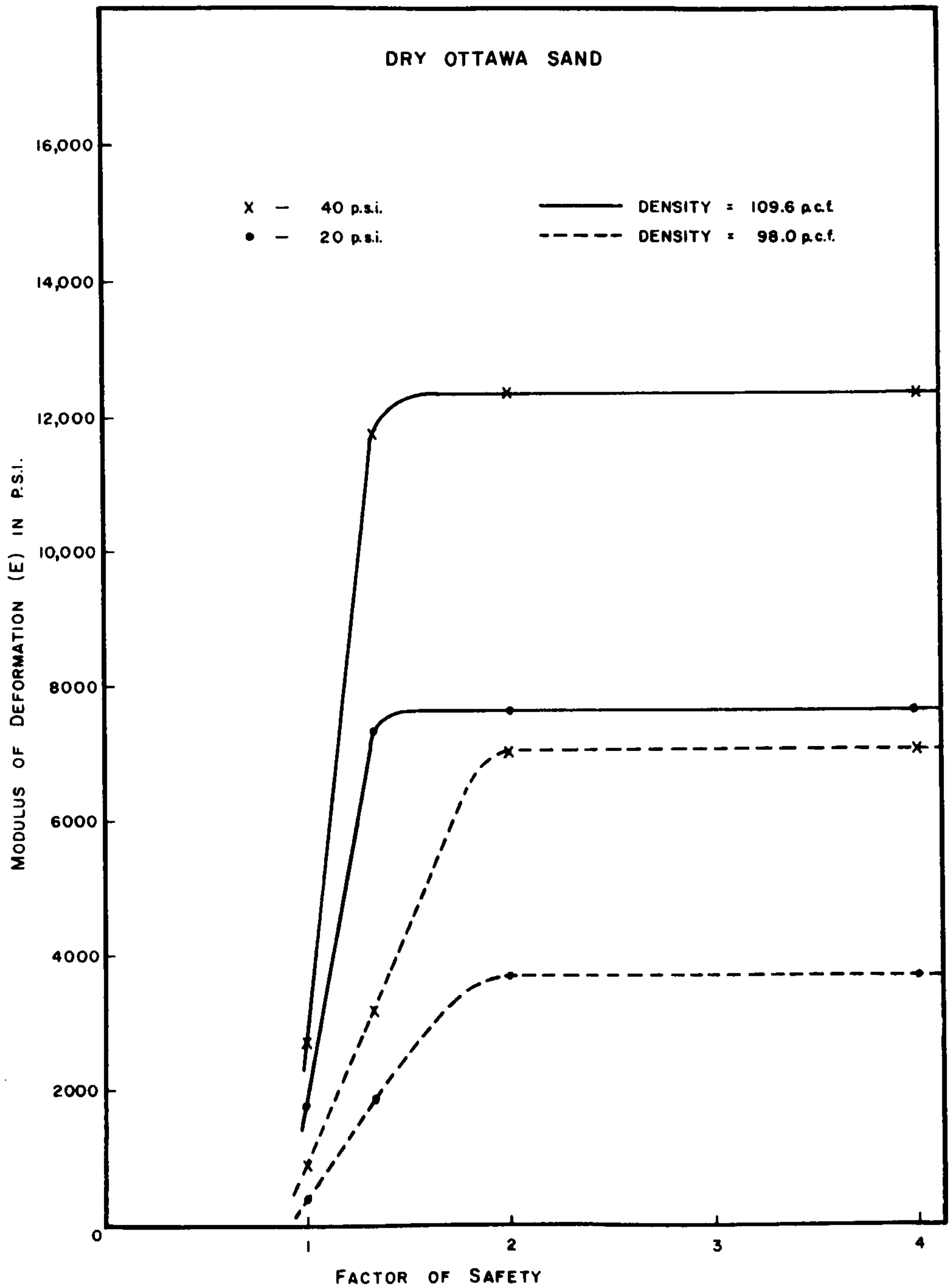


FIG. 16. EFFECT OF THE LEVEL OF STRESS ON THE MODULUS OF DEFORMATION

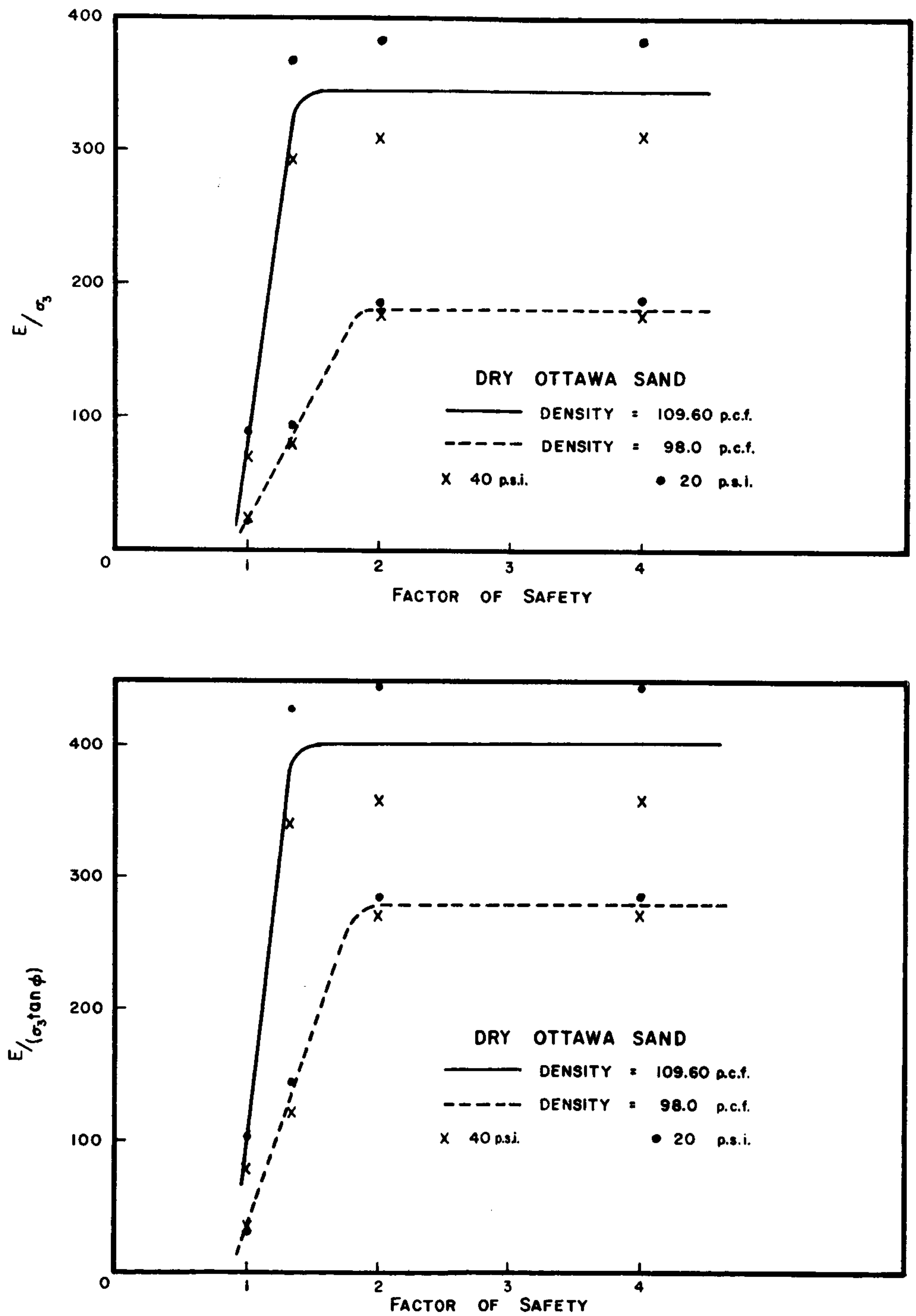


FIG. 17. NON-DIMENSIONAL CURVES

CHAPTER VI

TRIAXIAL COMPRESSION TESTS ON NORMALLY-CONSOLIDATED CLAYS

6.1 General

The stress-strain properties of two sands were presented in the previous two chapters. The next three chapters deal with such properties for clays. The present chapter includes the axial stress-strain behavior of three normally-consolidated clays. The investigated clays covered a wide range of plasticity and moisture content. The clay specimens were manufactured in the laboratory and were consolidated isotropically in triaxial cells. The author expects the general pattern of behavior of these specimens to be similar to the behavior of the same specimens consolidated anisotropically. The conventional quick-triaxial test was used in performing the present series of tests, under static load conditions.

The object of the tests was to investigate some of the variables presented in Art. 3.2. The stresses, strains, and modulus of deformation were studied. The factors investigated were the plasticity of the clay, moisture content, consolidation pressure, confining pressure, and level of stress and strain.

6.2 Clays Studied

Three clays, with different plasticity, were used in the laboratory study. The grain size distribution curves for these soils are given in Fig 3, and various other properties of the clays are presented in Table 16. All properties were determined using the generally accepted tests and procedures at the University of Texas soil mechanics laboratory (9).

TABLE 16

PROPERTIES OF THE STUDIED CLAYS

Soil Property	Taylor Marl No. 2	Taylor Marl No. 1	Vicksburg Silty Clay
Liquid Limit, per cent	73.1	48.7	34.0
Plastic Limit, per cent	20.0	21.0	22.0
Plasticity Index, per cent	53.1	27.7	12.0
Shrinkage Limit, per cent	14.6	14.0	----
Volumetric Shrinkage (per cent of dry volume)	109.0	37.5	----
Specific Gravity	2.75	2.75	2.70
Hygroscopic Moisture content, per cent	5.82	3.22	2.55

The first soil known as "Taylor Marl No. 2" is a yellow, calcareous clay obtained from an area in the vicinity of Austin, Texas. The soil comes from a geological formation known as Taylor Marl. The soil plots above the A-line on the Casagrande Plasticity Chart, and is classified as a high plasticity clay (CH) in accordance with the Unified Soil Classification System. The dry crushing strength is very high.

The second soil, identified as "Taylor Marl No. 1," was obtained from a cut about seven miles east of Austin, Texas. The geology of the region (31) indicates that it is a weathered Taylor Marl (found near the contact between Taylor Marl and Austin chalk). It has a whitish-yellow color and a high calcium carbonate content. It contains less colloidal clay than the normal Taylor Marl of Central Texas. It is classified as an inorganic clay of medium plasticity (CL). The dry crushing strength of the soil is high and the material slakes down rapidly in water.

The third soil is known as "Vicksburg Silty Clay" and was obtained from the vicinity of Vicksburg, Mississippi. It is a weathered leoss, that has a tan color. The soil is classified as an inorganic silty clay of low plasticity. The dry crushing strength is medium.

6.3 Preparation of Soil Samples

The clay specimens for this laboratory study were manufactured by a method of vacuum extrusion, used at The University of Texas (26). This method of preparation was selected to ensure clay specimens that have uniform density and moisture content, and a degree of saturation as near to a hundred per cent as possible.

The soil was first air dried, and then pulverized to pass a No. 40 U.S. Standard sieve. Sufficient water was than added to the soil to obtain the desired moisture content. The soil was thoroughly mixed to uniform consistency, sealed in containers, and allowed to temper in a moist room for about two weeks before extrusion (to allow uniform moisture distribution and stabilization throughout the material).

The moisture content of each soil was desired to be as close as possible to the liquid limit of the soil, to be able to obtain normally-consolidated samples under a wide range of consolidation pressures. At the same time the water content could not be too high, otherwise the extruded specimens would sag and deform excessively during handling. The upper limit of moisture content that could practically be used to extrude samples of any clay was arrived at by trial. The extruded specimens had about 58% moisture content in the case of Taylor Marl No. 2, 38% for Taylor Marl No. 1, and 28% for Vicksburg silty clay.

The vacuum extrusion machine, used in sample preparation, was manufactured by the International Clay Machinery Company. A simplified sketch of this device is shown in Fig 18. The soil is fed into a loading hopper at the rear of the first auger. This auger feeds the soil through a perforated plate between augers. This plate forms the soil into ribbons which are picked up by the second auger. A vacuum chamber, located above the second auger, places a vacuum of about 10 psi on the soil. This vacuum draws air bubbles out of the soil before it is forced through a 2.8 in. diameter circular die, to be transformed to the desired shape and size.

Soil specimens were cut in about 6.4 in. length, wrapped in Saran Wrap and sealed in jars. The jars were stored in a moist room for a period of at least two weeks to allow for curing and to eliminate any thixotropic effects.

The extruded specimens had practically the same density. The variation in moisture content from one specimen to the other, and within the same specimen,

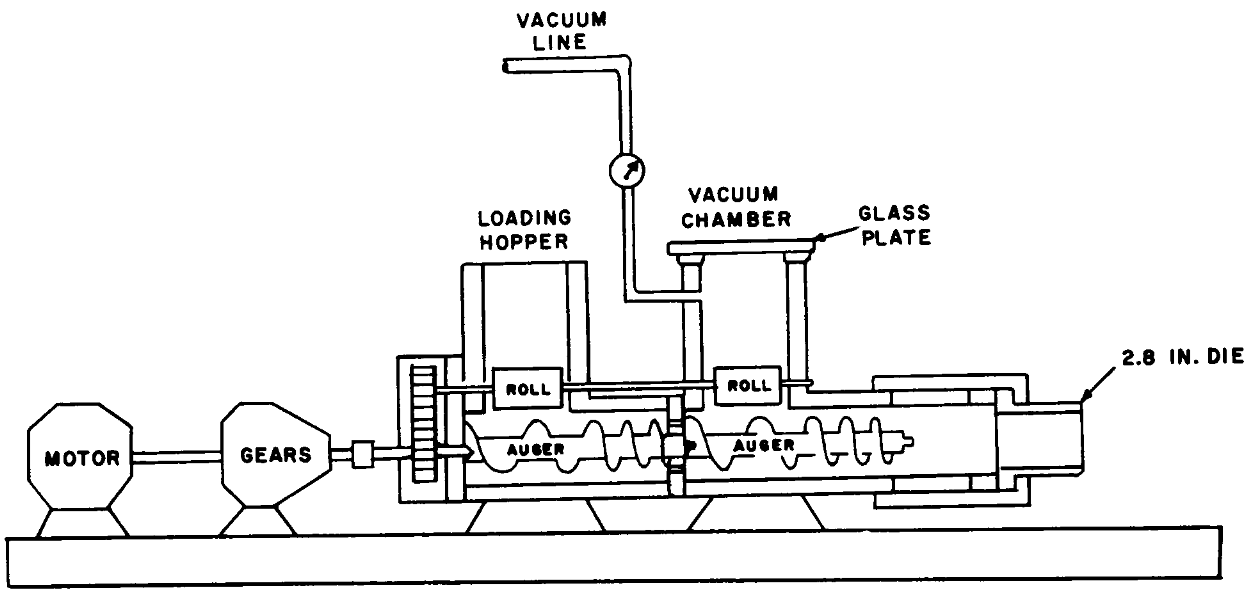


FIG. 18. VACUUM EXTRUSION MACHINE FOR SAMPLE PREPARATION

was found to be less than one per cent. The degree of saturation obtained by this method was about 98% in the average.

6.4 Test Equipment and Procedure

Normally-consolidated clay specimens were obtained by consolidating the extruded samples isotropically in triaxial cells, under the desired consolidation pressure. Depth was simulated by causing consolidation under different pressures. Clay specimens were consolidated under 10, 20, and 40 psi.

The extruded specimen, 2.8 in. diameter and 6.4 in. height, was removed from the moist room and prepared for consolidation. The specimen was put in a cylindrical split tube that had the same diameter and was 6.2 in. long. The ends of the sample were cut flush with the ends of the split tube, producing a specimen with similar dimensions. The specimen was then removed from the tube and transferred to the triaxial cell.

The triaxial cells used were the same as the one described in Art. 5.3 except for size of base, which in this case was 2.8 in. diameter. Top and bottom drainage were used to consolidate the sample. All drainage lines, porous stones, base, and top cap were saturated with de-aired, de-mineralized water. Saturated slit paper drains were also used to aid the drainage process. The top and bottom drainage outlets of the triaxial cell, controlled by valves, were connected to two graduated burettes filled with the same water, that had an oil layer at top to prevent evaporation. The burettes were used to give an indication of the progress of consolidation.

The specimen was placed on the base of the triaxial cell, that had a porous stone on. The top porous stone and cap were then placed on the sample, and the paper drains fitted around it. The membranes were then placed using a membrane stretcher, to reduce sample disturbance. Two membranes, with a coating of silicone grease in between, were used to prevent seepage. The membranes were 2.8 x 10 x 0.005 in. in size.

The membrane stretcher consisted of an aluminum tube, three in. inside diameter and seven in. long. The tube was fitted with a wire mesh liner, and had an outlet in the middle to be connected to a vacuum line. The vacuum caused the membrane to be held against the inside wall of the cylinder.

After fitting the membranes, O-rings were placed to seal the cap and base (two at each end). The drainage lines were connected and drainage valves opened. The triaxial cell was then put together, and the loading rod held in position. The air pressure, used for consolidation, was then applied.

The sample was left to consolidate until top and bottom drainage stopped, and the moisture content throughout the sample became uniform. The time necessary for the completion of the consolidation process varied for different clays and pressures. It ranged from four days for the Vicksburg silty clay under 10 psi to seven days for the Taylor Marl No. 2 under 40 psi. This time was determined through several preliminary tests in which moisture contents were checked to achieve uniform conditions in the soil sample. It was also checked roughly by watching the water level in the burettes.

At the end of consolidation, the specimen represented a clay that was normally-consolidated under a certain over-burden pressure, corresponding to the consolidation pressure used. The difference between such a specimen and field samples is believed to be the type of consolidation used.

The pressure was then reduced to zero, after closing the drainage valves, to simulate extraction of the sample from a deep strata. The specimen was taken out, its diameter measured, and its ends trimmed to give a length to diameter ratio equal to two. Moisture samples were taken from these trimmings.

The specimen was put once more in the triaxial cell, this time with only one membrane around, and no paper drains. Two smooth aluminum end plates were used instead of the porous stones to reduce friction. The drainage valves were closed after applying the same consolidation pressure (in some tests this pressure was less than the consolidation pressure). A quick triaxial test was performed on the sample, in the same manner described before for sands. The speed of testing was adjusted to give a rate of strain 1.5% per minute. At the end of the test the specimen was taken out and moisture samples secured from its top, center, and bottom. The maximum variation of moisture content within any sample did not exceed 0.5%.

As mentioned before, three clays were investigated. Samples of each clay were consolidated and tested under 10, 20, and 40 psi. Samples of Taylor Marl No. 1 and Vicksburg silty clay were also consolidated under 40 psi and tested under 20 psi, to study the effect of confining pressure.

6.5 Results and Discussion

The stress-strain curves for all normally-consolidated samples are given in Appendix C. Each test was repeated twice and the results averaged. The variation in the average moisture content between any two similar tests was always less than 0.6%

A. Axial stresses and strains

The properties of the three clays, as determined by quick triaxial tests at a confining pressure σ_3 equal to the consolidation pressure σ_c , are shown in Tables 17 and 18. It must be pointed out here that the stress-strain curves for the Vicksburg silty clay (Appendix C) do not indicate a maximum deviator stress. In such cases it is customary to choose the deviator stress at 0.2 in. per in. axial strain to arbitrarily represent the maximum

TABLE 17

Initial Strain ϵ_i , Strain at Maximum Stress ϵ_{100} , and

The Ratio of $\epsilon_i/\epsilon_{100}$ for Normally-Consolidated Clays

Consolidation Pressure σ_c (psi)	TAYLOR MARL NO. 2				TAYLOR MARL NO. 1				VICKSBURG SILTY CLAY			
	ϵ_i (in./in.)	ϵ_{100} (in./in.)	$\frac{\epsilon_i}{\epsilon_{100}}$	w%	ϵ_i (in./in.)	ϵ_{100} (in./in.)	$\frac{\epsilon_i}{\epsilon_{100}}$	w%	ϵ_i (in./in.)	ϵ_{75} (in./in.)	$\frac{\epsilon_i}{\epsilon_{100}}$	w%
10	0.0041	0.1180	0.0347	48.18	0.0038	1.1150	0.0330	33.11	0.0030	0.0940	0.0150	26.53
20	0.0053	0.1040	0.0510	44.18	0.0048	0.0996	0.0482	30.70	0.0035	0.0900	0.0175	25.07
40	0.0068	0.0960	0.0710	36.71	0.0062	0.0920	0.0674	25.62	0.0042	0.0862	0.0210	23.41

TABLE 18

Values of σ_{Δ_1} , $\sigma_{\Delta_{max}}$, $\sigma_{\Delta_{0.2}}$, $\sigma_{\Delta_1}/\sigma_{\Delta_{max}}$, and $\sigma_{\Delta_{0.2}}/\sigma_{\Delta_{max}}$ for Normally-Consolidated Clays

Consolidation Pressure σ_c (psi)	TAYLOR MARL NO. 2					TAYLOR MARL NO. 1					VICKSBURG SILTY CLAY		
	σ_{Δ_1} (psi)	$\sigma_{\Delta_{max}}$ (psi)	$\sigma_{\Delta_{0.2}}$ (psi)	$\frac{\sigma_{\Delta_1}}{\sigma_{\Delta_{max}}}$	$\frac{\sigma_{\Delta_{0.2}}}{\sigma_{\Delta_{max}}}$	σ_{Δ_1} (psi)	$\sigma_{\Delta_{max}}$ (psi)	$\sigma_{\Delta_{0.2}}$ (psi)	$\frac{\sigma_{\Delta_1}}{\sigma_{\Delta_{max}}}$	$\frac{\sigma_{\Delta_{0.2}}}{\sigma_{\Delta_{max}}}$	σ_{Δ_1} (psi)	$\sigma_{\Delta_{max}}$ $(\sigma_{\Delta_{0.2}})$ (psi)	$\frac{\sigma_{\Delta_1}}{\sigma_{\Delta_{max}}}$
10	3.62	7.38	6.91	0.491	0.936	3.80	6.68	6.23	0.569	0.933	4.25	8.46	0.502
20	5.62	10.28	9.57	0.547	0.931	5.80	8.74	8.12	0.663	0.929	6.00	11.20	0.5357
40	11.52	18.86	17.41	0.611	0.923	14.00	20.08	18.20	0.697	0.906	12.00	23.20	0.560

condition.

In all quick tests run on clay, in this study, the cohesion of the clay c is taken to be half the maximum deviator stress, whether this maximum is represented by a peak value as in the case of Taylor Marl clays or by the value at 0.2 in per in. strain for the Vicksburg silty clay.

The effect of simulated depth involves both pressure and moisture content. For normally-consolidated clays there is a definite relation between the consolidation pressure increases and moisture content. As depth increases, the consolidation pressure increases and moisture content decreases.

Table 17 shows that as the consolidation pressure increased (and moisture content decreased) the initial strain ϵ_i increased, the strain at the maximum deviator stress ϵ_{100} decreased, and the ratio $\epsilon_i/\epsilon_{100}$ increased. It is also observed that for the same consolidation pressure the value of ϵ_i, and the ratio $\epsilon_i/\epsilon_{100}$ increased with the increase in the plasticity index (P.I.) of the soil.

The stress at the end of the linear portion of the stress-strain curve σ_{Δ_i}, the maximum deviator stress $\sigma_{\Delta_{max}}$, the stress at 0.2 in. per in. strain $\sigma_{\Delta_{0.2}}$, and the ratio $\sigma_{\Delta_i}/\sigma_{\Delta_{max}}$ for any clay increased as the consolidation pressure increased, while the ratio $\sigma_{\Delta_{0.2}}/\sigma_{\Delta_{max}}$ decreased. For the clays with stress-strain curves that peaked to a maximum stress (Taylor Marl. No. 1 and 2) the value of σ_{Δ_i} and $\sigma_{\Delta_i}/\sigma_{\Delta_{max}}$ decreased with the increase in the plasticity index, while the ratio $\sigma_{\Delta_{0.2}}/\sigma_{\Delta_{max}}$ increased, at any consolidation pressure.

From the presented data it can be concluded that with the increase in depth (higher σ_0 and lower $w\%$) or decrease in plasticity index normally-consolidated clays generally exhibit stress-strain curves that are linear to a greater stress and strain level, peak more, and decrease to a lower stress value after the maximum.

Table 19 includes the results of tests run on specimens of two clays normally-consolidated under 40 psi then tested with a confining pressure σ_3 equal to 20 psi. The results indicate that the reduced confining pressure had some effect on axial strains. The strains at any level of stress are generally greater for the lower confining pressures.

The effect of lowering the confining pressure on the maximum deviator stress of Taylor Marl No. 1 specimens was negligible. This means that the Mohr envelope for quick tests on specimens of this clay, consolidated under the same pressure and tested at varying values of σ_3, will be a horizontal line with $\phi = 0$ and $\tau = c = 1/2\ \sigma_{\Delta_{max}}$. The lower confining pressure gave a slightly lower value of $\sigma_{\Delta_{max}}$ for the Vicksburg silty clay samples. This indicates that the Mohr envelope in this case will show a very small ϕ value.

A correlation between the moisture content, cohesion, and plasticity of the clay is presented in Appendix D. This correlation is given only as evidence of the trend developed from a limited number of tests conducted in this study. More research is definitely required however, before any final correlation is reached.

Table 19 also shows that as the confining pressure decreased, σ_{Δ_1} and $\sigma_{\Delta_1}/\sigma_{\Delta_{max}}$ decreased, and $\sigma_{\Delta_{0.2}}$ and $\sigma_{\Delta_{0.2}}/\sigma_{\Delta_{max}}$ increased. It must be recognized here that the reported effects of confining pressure are based on the results of a few tests performed to give only a preliminary indication. More tests are required in this respect.

B. Modulus of Deformation

In the following pages a number of tables and graphs will be presented

TABLE 19

AXIAL STRESS-STRAIN BEHAVIOR OF NORMALLY-CONSOLIDATED CLAYS TESTED AT OTHER THAN THE CONSOLIDATION PRESSURE (σ_c = 40 psi, σ_3 = 20 psi)

A - TAYLOR MARL NO. 1

	σ_Δ (psi)	ϵ (in/in)	E (psi)	E/σ_c	E/σ_3	E/c	$\sigma_\Delta/\sigma_{\Delta_{max}}$	ϵ/ϵ_{max}
At End of straight-line Portion	13.18	0.0062	2136	53.4	106.8	214.0	0.6597	0.0569
At 50% of $\sigma_{\Delta_{max}}$	9.99	0.0047	2136	53.4	106.8	214.0	0.5000	0.0432
At $\sigma_{\Delta_{max}}$	19.98	0.1084	184	4.6	9.2	18.4	1.0000	1.0000
At ϵ = 0.2 in/in.	19.12	0.2000	96	2.4	4.8	9.6	0.9570	1.8450

B - VICKSBURG SILTY CLAY

	σ_Δ (psi)	ϵ (in/in)	E (psi)	E/σ_c	E/σ_3	E/c	$\sigma_\Delta/\sigma_{\Delta_{max}}$	ϵ/ϵ_{max}
At End of straight-line Portion	11.8 4	0.0040	2960	74.0	148.0	266.0	0.5314	0.0200
At 50% of $\sigma_{\Delta_{max}}$	11.14	0.0038	2960	74.0	148.0	266.0	0.5000	0.0188
At 75% of $\sigma_{\Delta_{max}}$	16.71	0.0900	186	4.7	9.3	16.7	0.7500	0.4500
At $\sigma_{\Delta_{max}}$ (taken at ϵ = 0.2 in/in.	22.28	0.2000	111	2.8	5.6	10.0	1.0000	1.0000

that show the variation of the modulus of deformation of normally-consolidated clays with the consolidation pressure, moisture content, confining pressure, clay plasticity, and level of stress and strain.

Table 20 gives the initial modulus of deformation E_i , Table 21 the mosulus at half the maximum deviator stress E_{50} , and Table 22, the modulus at maximum stress E_{100} . The values of the modulus in these tables are also divided by the consolidation pressure σ_c , and by the cohesion of the clay c .

The initial modulus E_i , for any clay, increased with the increase in consolidation pressure. Also as the plasticity of the clay decreased the value of E_i became higher.

The effect of the level of strain is shown in Figs 19 and 20. It can be seen that the modulus remained constant for a small value of strain, then decreased sharply with the increase in strain. This decrease became much less at the higher values of strain, where the curves flatten considerably. The increase in E . at the same strain level, was more for the higher range of σ_c than for the lower range.

Figure 21 indicates that for all three clays, and at any consolidation pressure, the modulus of deformation remained constant to a deviator stress above half the maximum value and then decreased rapidly with the increase in the level of stress.

To present the modulus of deformation of the clay in a non-dimensional form, the modulus is divided by the consolidation pressure, as shown in Fig 22. It is obvious from this figure that the ratio E/σ_c , at any level of stress, decreased with the increase in σ_c . However, this decrease was less for the higher pressure range. Actually one average value of E/σ_c could be considered for the range of pressures between 20 and 40 psi, at any

TABLE 20

E_i , INITIAL MODULUS OF DEFORMATION FOR

NORMALLY-CONSOLIDATED CLAYS

Consolidation Pressure σ_c (psi)	TAYLOR MARL NO. 2				TAYLOR MARL NO. 1				VICKSBURG SILTY CLAY			
	E_i (psi)	w%	$\frac{E_i}{\sigma_c}$	$\frac{E_i}{c}$	E_i (psi)	w%	$\frac{E_i}{\sigma_c}$	$\frac{E_i}{c}$	E_i (psi)	w%	$\frac{E_i}{\sigma_c}$	$\frac{E_i}{c}$
10	883	48.18	88.3	239.3	1000	33.11	100.0	299.0	1417	26.53	141.7	335.0
20	1060	44.18	53.0	206.0	1208	30.70	60.4	276.0	1714	25.07	85.7	306.1
40	1694	36.71	42.4	179.6	2258	25.62	56.5	224.9	3095	23.41	77.4	266.8

TABLE 21

E_{50} , MODULUS OF DEFORMATION AT HALF THE MAXIMUM DEVIATOR

STRESS FOR NORMALLY-CONSOLIDATED CLAYS

Consolidation Pressure σ_c (psi)	TAYLOR MARL NO. 2				TAYLOR MARL NO. 1				VICKSBURG SILTY CLAY			
	E_{50} (psi)	w%	$\frac{E_{50}}{\sigma_c}$	$\frac{E_{50}}{c}$	E_{50} (psi)	w%	$\frac{E_{50}}{\sigma_c}$	$\frac{E_{50}}{c}$	E_{50} (psi)	w%	$\frac{E_{50}}{\sigma_c}$	$\frac{E_{50}}{c}$
10	820	48.18	82.0	222.2	1000	33.11	100.0	299.0	1417	26.53	141.7	335.0
20	1060	44.18	53.0	206.0	1208	30.70	60.4	276.0	1714	25.07	85.7	306.1
40	1694	36.71	42.4	179.6	2258	25.62	56.5	224.9	3095	23.41	77.4	266.8

TABLE 22

E_{100} , MODULUS OF DEFORMATION AT MAXIMUM DEVIATOR

STRESS FOR NORMALLY-CONSOLIDATED CLAYS

Consolidation Pressure σ_c (psi)	TAYLOR MARL NO. 2				TAYLOR MARL NO. 1				VICKSBURG SILTY CLAY			
	E_{100} (psi)	w%	$\frac{E_{100}}{\sigma_c}$	$\frac{E_{100}}{c}$	E_{100} (psi)	w%	$\frac{E_{100}}{\sigma_c}$	$\frac{E_{100}}{c}$	E_{100} (psi)	w%	$\frac{E_{100}}{\sigma_c}$	$\frac{E_{100}}{c}$
10	62.5	48.18	6.25	16.95	58.1	33.11	5.81	17.40	42.3	26.53	4.23	10.00
20	99.0	44.18	4.95	19.20	87.8	30.70	4.39	20.10	56.0	25.07	2.80	10.00
40	196.5	36.71	4.91	20.90	218.0	25.62	5.45	20.90	116.0	23.41	2.90	10.00

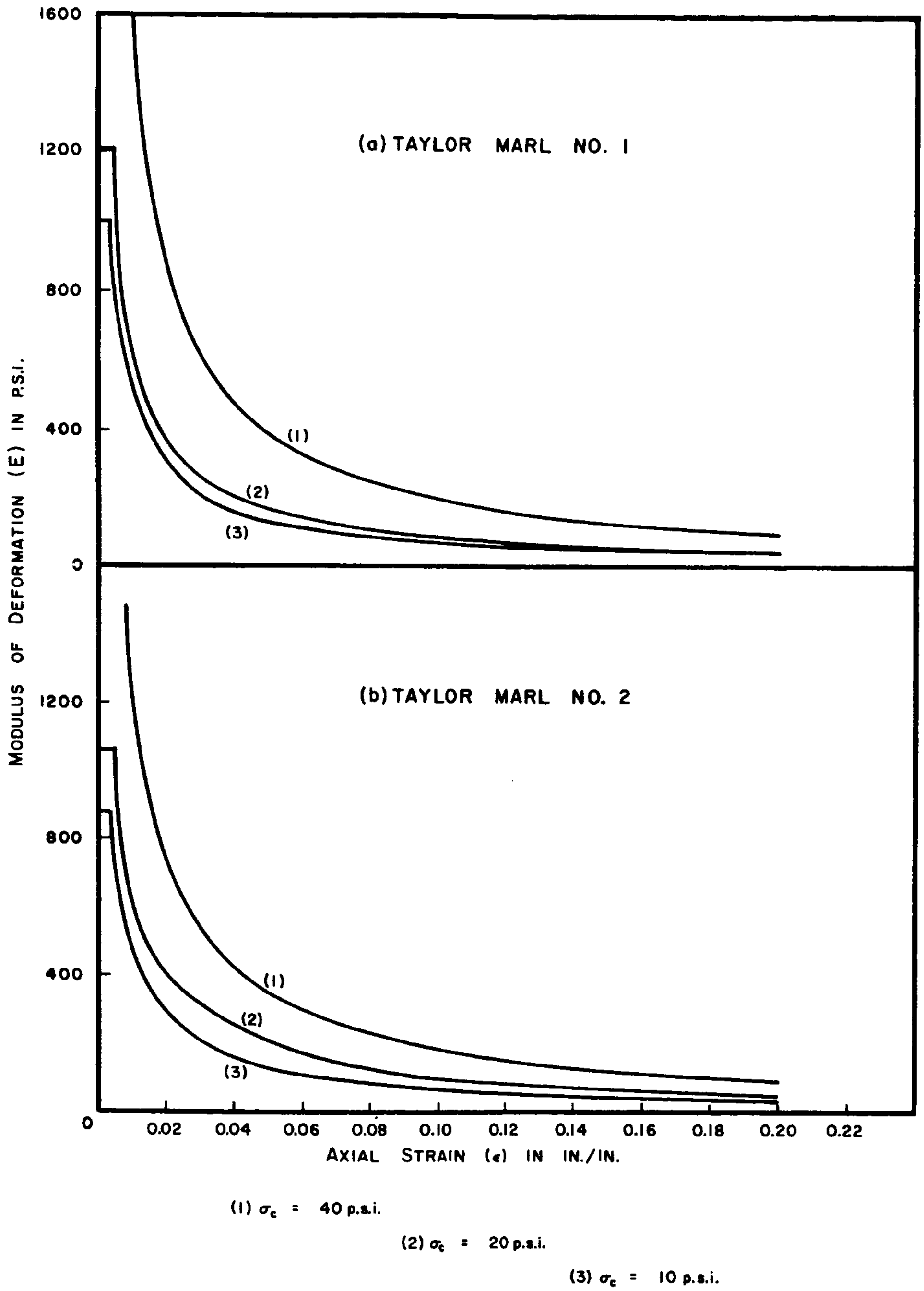


FIG.19. MODULUS OF DEFORMATION CURVES FOR NORMALLY CONSOLIDATED SAMPLES

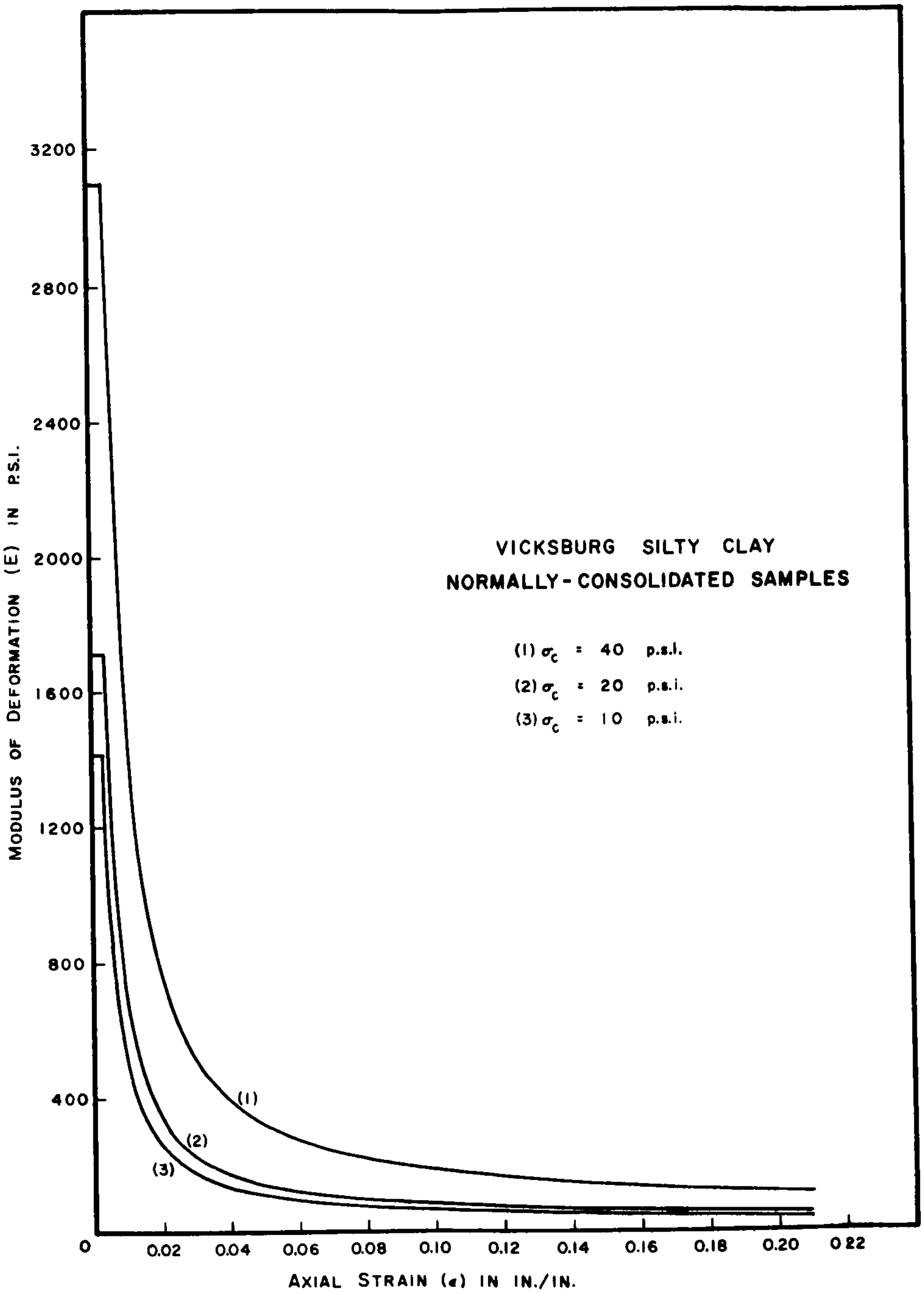


FIG. 20. MODULUS OF DEFORMATION VS. AXIAL STRAIN

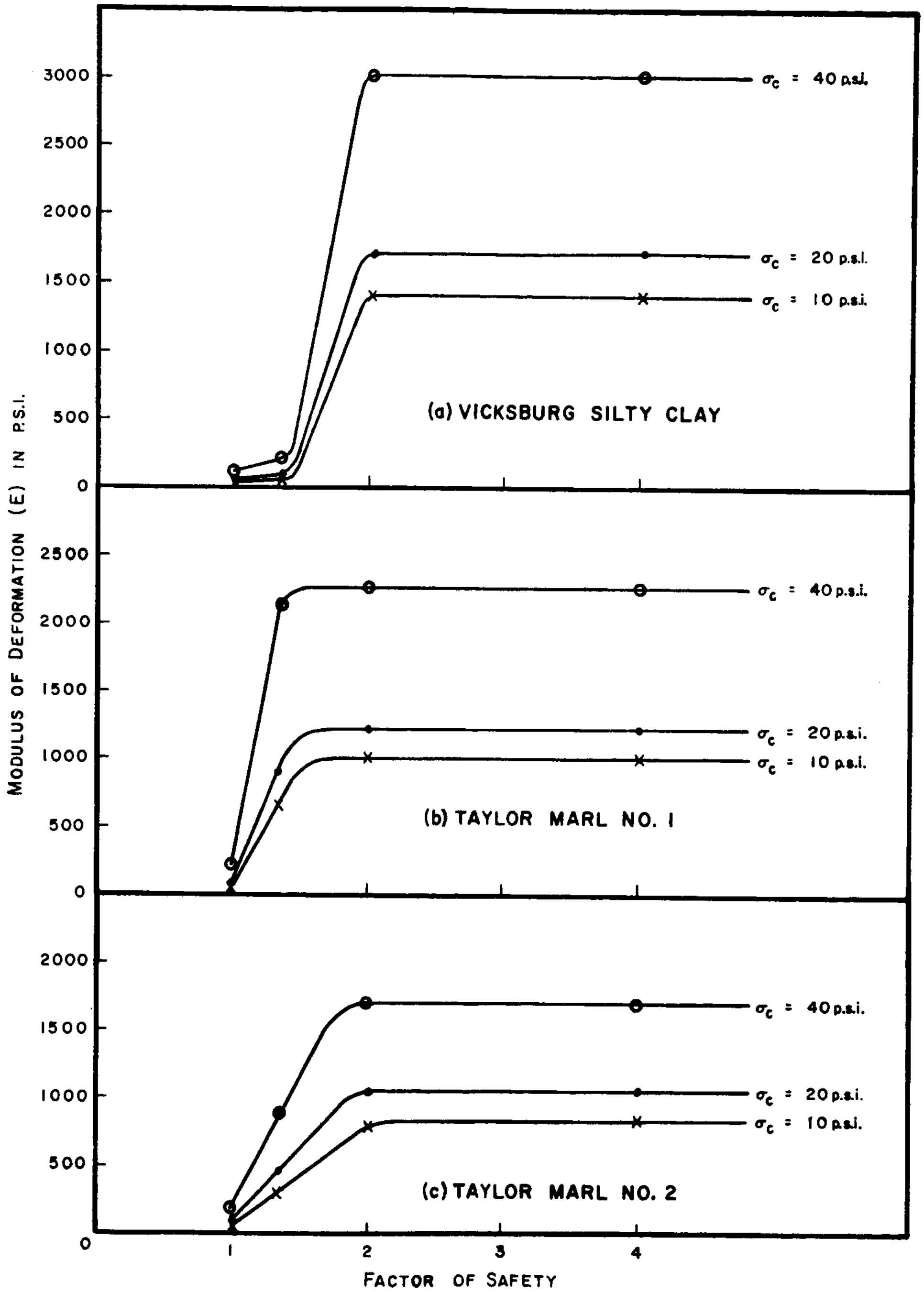


FIG. 21. EFFECT OF THE LEVEL OF STRESS ON MODULUS OF DEFORMATION OF NORMALLY-CONSOLIDATED CLAYS

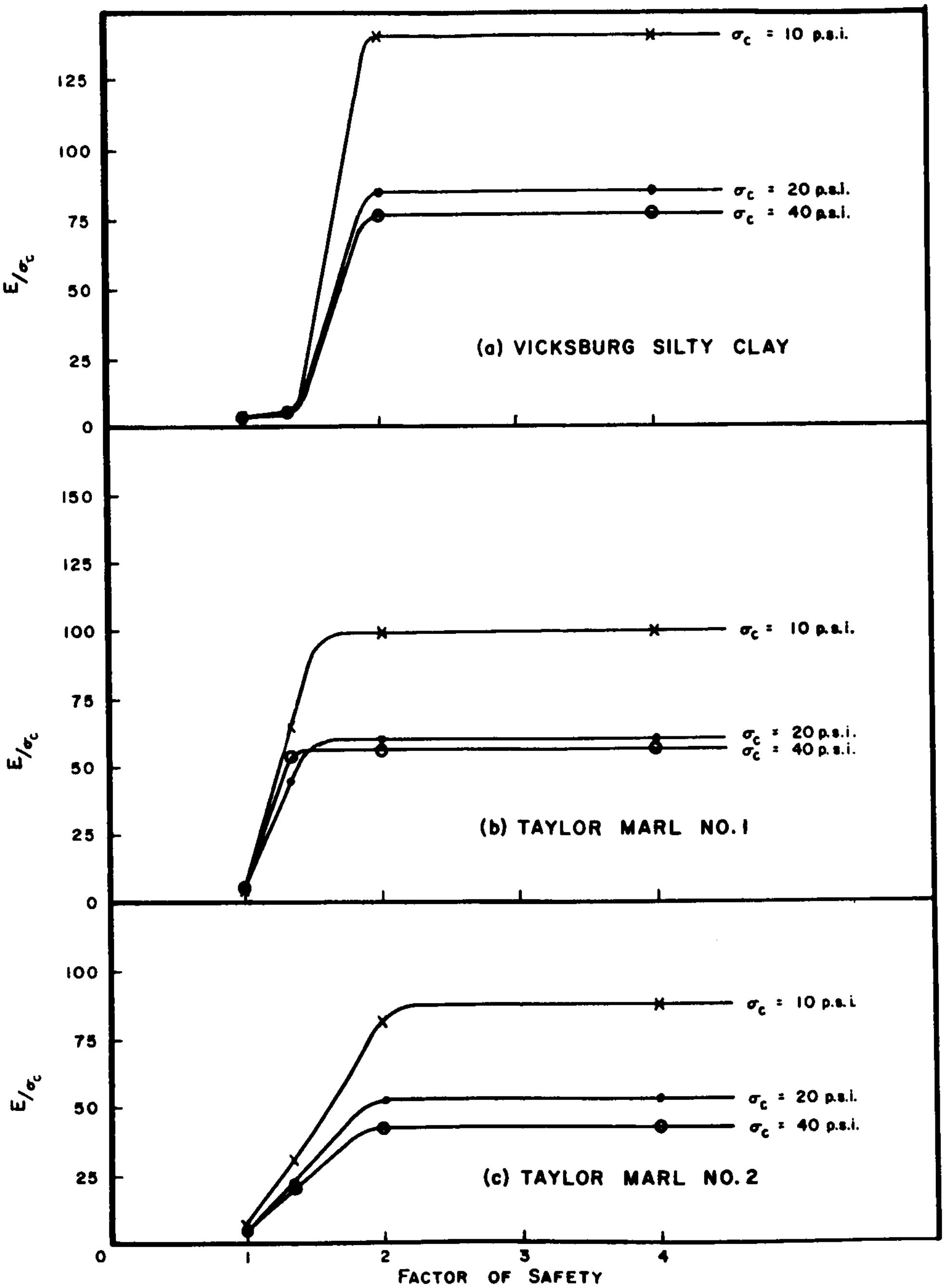


FIG. 22. VARIATION OF THE RATIO OF MODULUS OF DEFORMATION (E) TO THE CONSOLIDATION PRESSURE (σ_c) WITH THE LEVEL OF STRESS FOR NORMALLY-CONSOLIDATED CLAYS

level of stress. It can also be seen that the variation of E/σ_c with consolidation pressure was much less at the higher stress levels than at the lower levels. The value of E/σ_c vaired with the level of stress, for any clay and consolidation pressure.

In Fig 23 the ratio of the modulus of deformation to the cohesion of the clay is presented. This non-dimensional ratio, at any value of σ_c , varied with the level of stress as shown in figure. It also decreased with the increase in σ_c , at any level of stress. This decrease, however, was less at the higher stress levels. The decrease in E/c at the same F.S., with the increase in σ_c from 10 to 20 psi, or from 20 to 40 psi, was generally in the same order of magnitude.

From the presented data it can be concluded that the plasticity index of the clay has an effect on the modulus of deformation. This effect is demonstrated in Fig 24, for E_{50} . It can be seen that the modulus of deformation, the ratio E/σ_c and E/c (at the same level of stress and consolidation pressure) increased with the decrease in plasticity index. The modulus of deformation, at the same strain level and consolidation pressure, was also greater for Taylor Marl No. 1 than for Taylor Marl No. 2 specimens.

The effect of confining pressure can be observed from Table 19 and Fig 25. As σ_3 decreased, the modulus of deformation E/σ_c , and E/c (at the same level of stress) decreased. Also the modulus at any level of strain decreased.

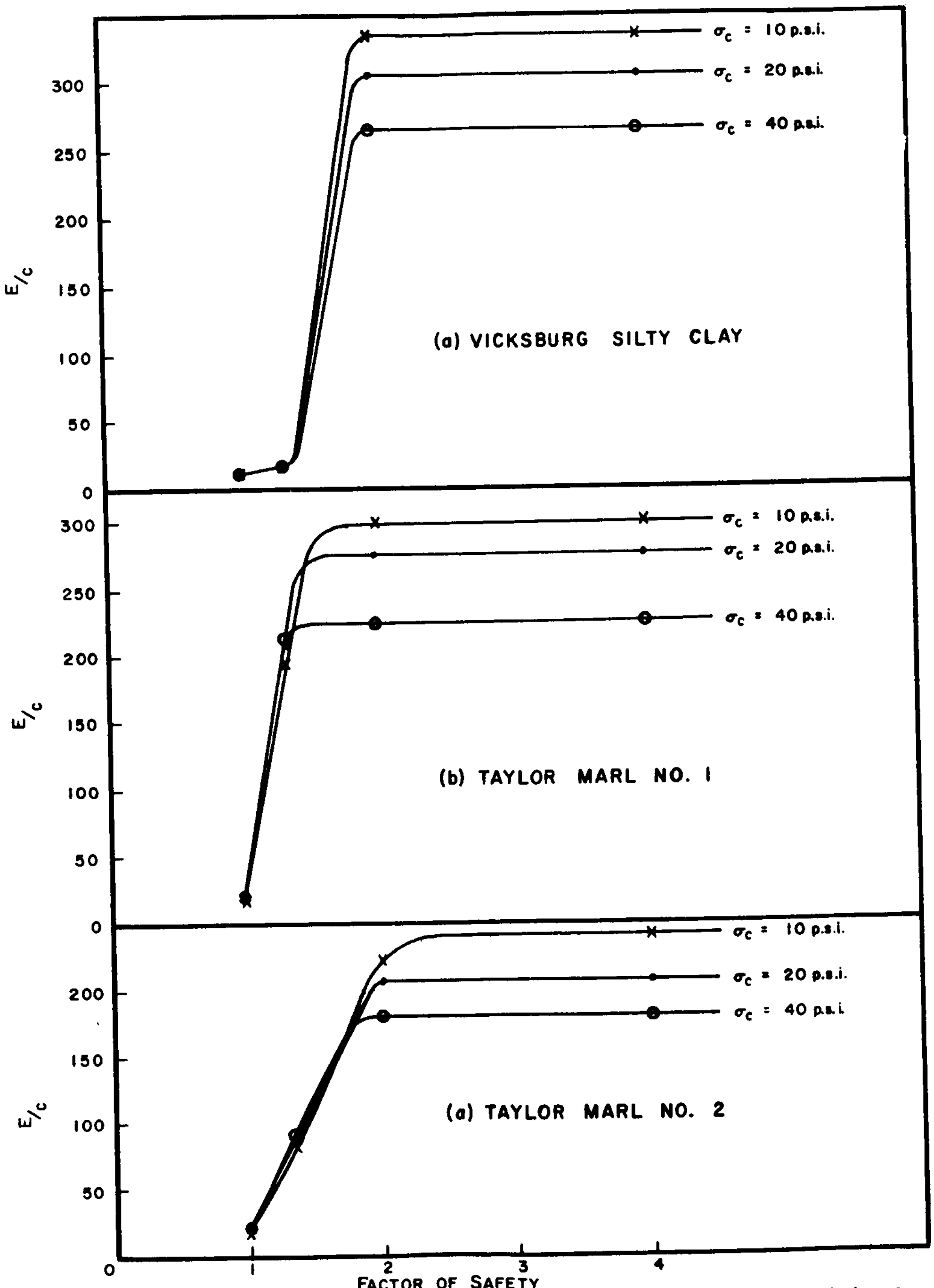
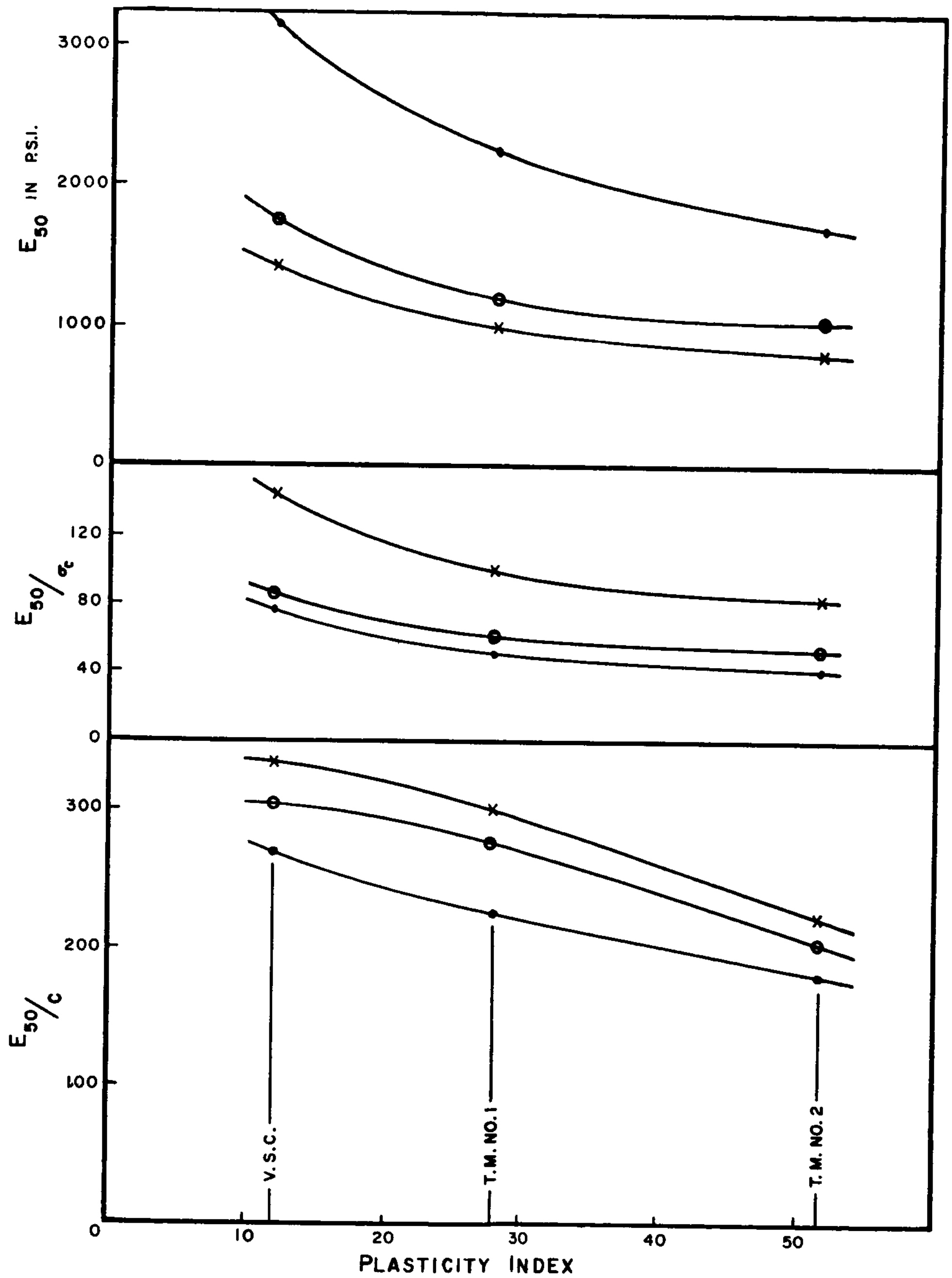


FIG. 23. VARIATION OF THE RATIO OF MODULUS OF DEFORMATION (E) TO COHESION (c) WITH THE LEVEL OF STRESS FOR NORMALLY-CONSOLIDATED CLAYS



• σ_c = 40 p.s.i. ● σ_c = 20 p.s.i. X σ_c = 10 p.s.i.

FIG. 24. EFFECT OF PLASTICITY ON THE MODULUS OF DEFORMATION OF NORMALLY-CONSOLIDATED CLAYS

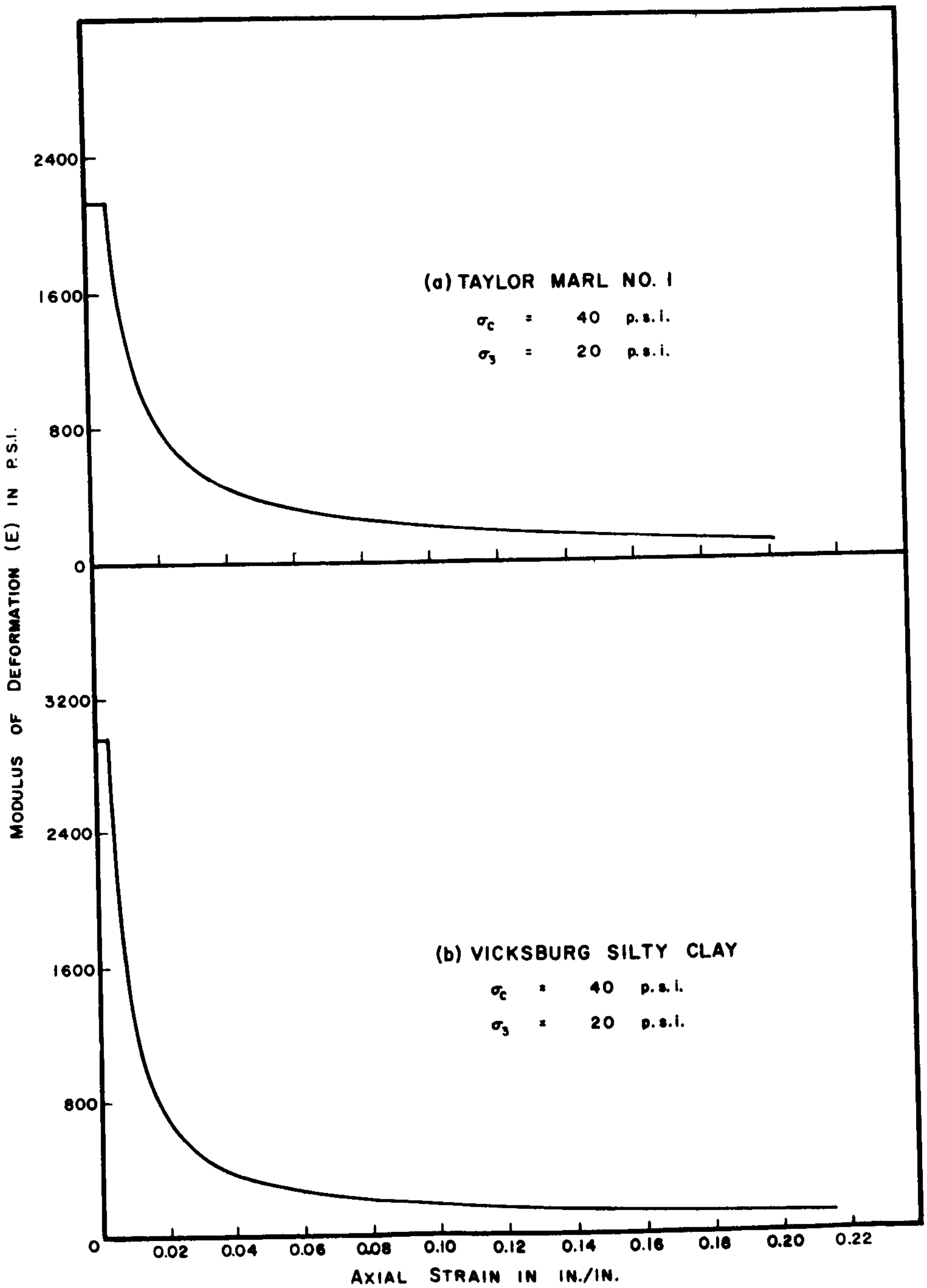


FIG. 25. EFFECT OF CONFINING PRESSURE ON MODULUS OF DEFORMATION OF NORMALLY-CONSOLIDATED CLAYS

CHAPTER VII

TRIAXIAL COMPRESSION TESTS ON OVER-CONSOLIDATED CLAYS

7.1 General

The effect of stress-history on the axial stress-strain characteristics of clays is investigated in this chapter. Specimens of two clays having the same over-consolidation pressure σ_0 , and varying over-consolidation ratios were tested (O.C.R. = σ_0/σ_c) . The investigated clays were Taylor Marl No. 1 and 2. The specimens were extruded as described in Art. 6.3, and then over-consolidated in the laboratory as will be discussed in the next article. The test equipment and set up were the same as in the case of normally consolidated clays.

The object of this series of tests was to determine the effect of the over-consolidation ratio on the axial stresses, strains, and modulus of deformation of clays having the same σ_0 , also to compare the axial stress-strain behavior of such clays with that of normally-consolidated specimens.

It must be pointed out that more research is required to investigate fully the behavior of over-consolidated clays and study the various factors influencing it.

7.2 Test Procedure

The test procedure is similar to the one used for normally-consolidated clays except for one thing. Extruded specimens of Taylor Marl No. 1 and 2 were consolidated in triaxial cells under 40 psi, representing the over-consolidation pressure, as described before in Art. 6.4. At the end of this consolidation period (under σ_0) the consolidation pressure was reduced to 20 psi in some cases, and to 10 psi in others (representing the consolidation pressure σ_c). The specimens were left under this new pressure until stability was achieved and the moisture content became uniform throughout the specimen. It was found, through preliminary tests and by watching the level of water in the burettes, that a period equal to the original consolidation period (under σ_0) was quite sufficient to reach this stable condition.

After the completion of the second consolidation process (under σ_c), the specimen was taken out and the test continued in exactly the same manner as given in Art. 6.4. The specimens at this point represented an over-consolidated clay with σ_0 equal to 40 psi and σ_c equal to either 20 or 10 psi. The over-consolidation ratio is defined as the ratio of the over-consolidation pressure σ_0 to the existing consolidation (or over-burden) pressure σ_c . Specimens with σ_c equal to 20 psi, represent an O.C.R. of 2. Those with σ_c equal to 10 psi have an O.C.R. of 4. Normally consolidated clays have an O.C.R. equal to one.

Specimens of both Taylor Marl No. 1 and 2 were tested under O.C.R. equal to one, two, and four. Quick triaxial tests were performed on the samples with the confining pressure equal to σ_c .

7.3 Results and Discussion

The stress-strain curves of over-consolidation clays (O.C.C.) are given in Appendix E. The results are also summarized in Tables 23 and 24.

The maximum variation in moisture content of the top, center, and bottom of all specimens was equal to 0.42%.

A. Axial Stresses and Strains

For the same clay, the initial strain ϵ_i increased with the decrease

TABLE 23

STRESSES, STRAINS, AND MODULUS OF DEFORMATION FOR OVER-CONSOLIDATED CLAYS (σ_o = 40 psi)

(a) At the End of the Straight Line Portion of the Stress-Strain Curve

O.C.R.	TAYLOR MARL NO. 1								TAYLOR MARL NO. 2							
	ϵ_1 (in/in)	σ_{Δ_1} (psi)	E_1 (psi)	$\frac{E_1}{\sigma_o}$	$\frac{E_1}{\sigma_c}$	$\frac{E_1}{c}$	$\frac{\sigma_{\Delta_1}}{\sigma_{\Delta_{max}}}$	$\frac{\epsilon_1}{\epsilon_{100}}$	ϵ_1 (in/in)	σ_{Δ_1} (psi)	E_1 (psi)	$\frac{E_1}{\sigma_o}$	$\frac{E_1}{\sigma_c}$	$\frac{E_1}{c}$	$\frac{\sigma_{\Delta_1}}{\sigma_{\Delta_{max}}}$	$\frac{\epsilon_1}{\epsilon_{100}}$
1 (N.C.C.)	0.0062	14.00	2258	56.5	56.5	224.9	0.6970	0.0674	0.0068	11.52	1694	42.4	42.4	179.6	0.6110	0.0710
2	0.0060	12.49	2082	52.0	104.1	229.8	0.6895	0.0638	0.0063	8.69	1379	34.5	69.0	188.4	0.5940	0.0626
4	0.0055	9.61	1747	43.7	174.7	243.7	0.6700	0.0568	0.0057	6.33	1110	27.8	111.0	182.3	0.5200	0.0535

(b) At Half the Maximum Deviator Stress

O.C.R.	TAYLOR MARL NO. 1								TAYLOR MARL NO. 2							
	ϵ_{50} (in/in)	$\sigma_{\Delta_{50}}$ (psi)	E_{50} (psi)	$\frac{E_{50}}{\sigma_o}$	$\frac{E_{50}}{\sigma_c}$	$\frac{E_{50}}{c}$	w%	σ_c (psi)	ϵ_{50} (in/in)	$\sigma_{\Delta_{50}}$ (psi)	E_{50} (psi)	$\frac{E_{50}}{\sigma_o}$	$\frac{E_{50}}{\sigma_c}$	$\frac{E_{50}}{c}$	w%	σ_c (psi)
1 (N.C.C.)	0.00445	10.04	2258	56.5	56.5	224.9	25.62	40	0.0056	9.43	1694	42.4	42.4	179.6	36.71	40
2	0.0045	9.06	2082	52.0	104.1	229.8	26.96	20	0.0053	7.27	1379	34.5	69.0	188.4	39.65	20
4	0.0041	7.17	1747	43.7	174.7	243.7	28.43	10	0.0055	6.09	1110	27.8	111.0	182.3	43.10	10

TABLE 24

STRESSES, STRAINS, AND MODULUS OF DEFORMATION

FOR OVER-CONSOLIDATED CLAYS (σ_o = 40 psi)

(a) At Maximum Deviator Stress

O.C.R.	TAYLOR MARL NO. 1						TAYLOR MARL NO. 2					
	ϵ_{100} (in/in)	$\sigma_{\Delta_{max}}$ (psi)	E_{100} (psi)	$\frac{E_{100}}{\sigma_o}$	$\frac{E_{100}}{\sigma_c}$	$\frac{E_{100}}{c}$	ϵ_{100} (in/in)	$\sigma_{\Delta_{max}}$ (psi)	E_{100} (psi)	$\frac{E_{100}}{\sigma_o}$	$\frac{E_{100}}{\sigma_c}$	$\frac{E_{100}}{c}$
1 (N.C.C.)	0.0920	20.08	218.0	5.5	5.5	20.9	0.0960	18.86	196.5	4.9	4.9	20.9
2	0.0941	18.12	193.0	4.8	9.7	21.3	0.1006	14.54	144.5	3.6	7.2	19.7
4	0.0968	14.34	148.0	3.7	14.8	20.6	0.1066	12.18	114.0	2.9	11.4	18.7

(b) At an Axial Strain Equal to 0.2 In. per In.

O.C.R.	TAYLOR MARL NO. 1						TAYLOR MARL NO. 2					
	$\sigma_{\Delta_{0.2}}$ (psi)	$E_{0.2}$ (psi)	$\frac{E_{0.2}}{\sigma_o}$	$\frac{E_{0.2}}{\sigma_c}$	$\frac{E_{0.2}}{c}$	$\frac{\sigma_{\Delta_{0.2}}}{\sigma_{\Delta_{max}}}$	$\sigma_{\Delta_{0.2}}$ (psi)	$E_{0.2}$ (psi)	$\frac{E_{0.2}}{\sigma_o}$	$\frac{E_{0.2}}{\sigma_c}$	$\frac{E_{0.2}}{c}$	$\frac{\sigma_{\Delta_{0.2}}}{\sigma_{\Delta_{max}}}$
1 (N.C.C.)	18.2	91.0	2.28	2.28	8.75	0.906	17.4	87.0	2.18	2.18	9.22	0.923
2	16.5	82.5	2.06	4.13	9.10	0.911	13.6	67.8	1.70	3.39	9.26	0.926
4	13.2	66.1	1.65	6.61	9.22	0.922	11.4	56.9	1.42	5.69	9.34	0.934

in the O.C.R. Increased O.C.R. means decreased σ_o and increased moisture content of the clay. The values of ϵ_{50} and ϵ_{100} decreased as the O.C.R. decreased.

For all specimens the stress, at the same level of strain, increased with the decrease in the O.C.R. Also σ_{Δ_1} , $\sigma_{\Delta_{max}}$, and $\sigma_{\Delta_{0.2}}$ all increased with decreased O.C.R.

The ratios $\sigma_{\Delta_1}/\sigma_{\Delta_{max}}$ and $\epsilon_1/\epsilon_{100}$, for any clay, increased as the O.C.R. decreased, while the ratio $\sigma_{\Delta_{0.2}}/\sigma_{\Delta_{max}}$ decreased.

The effect of the clay plasticity is demonstrated by comparing the results of Taylor Marl No. 1 and 2, shown in Tables 23 and 24 and in Appendix E. It can be seen that, at the same O.C.R., ϵ_1 increased and σ_{Δ_1} decreased with the increase in the plasticity index. Also the stress, at the same strain and O.C.R., decreased as the P.I. increased. The increase in the plasticity index caused a decrease in the value of $\sigma_{\Delta_{max}}$, $\sigma_{\Delta_{0.2}}$, and the ratio $\sigma_{\Delta_1}/\sigma_{\Delta_{max}}$, and an increase in the value of ϵ_{100} and the ratio $\sigma_{\Delta_{0.2}}/\sigma_{\Delta_{max}}$ (at the same O.C.R.).

The results of over-consolidated clay samples will now be compared with those of tests on normally-consolidated specimens with the same σ_o , which were presented in Art. 6.5. It was observed that the value of ϵ_1 , $\epsilon_1/\epsilon_{100}$, σ_{Δ_1} , $\sigma_{\Delta_1}/\sigma_{\Delta_{max}}$, $\sigma_{\Delta_{max}}$, and $\sigma_{\Delta_{0.2}}$ are all smaller for normally consolidated samples than for over-consolidated specimens, with the same σ_o . Also the value of ϵ_{100} , w% and $\sigma_{\Delta_{0.2}}/\sigma_{\Delta_{max}}$ are larger for N.C.C. than for O.C.C.

B. Modulus of Deformation

The effect of the level of strain on the modulus of deformation of over-consolidated samples is presented in Fig 26. The variation of the modulus

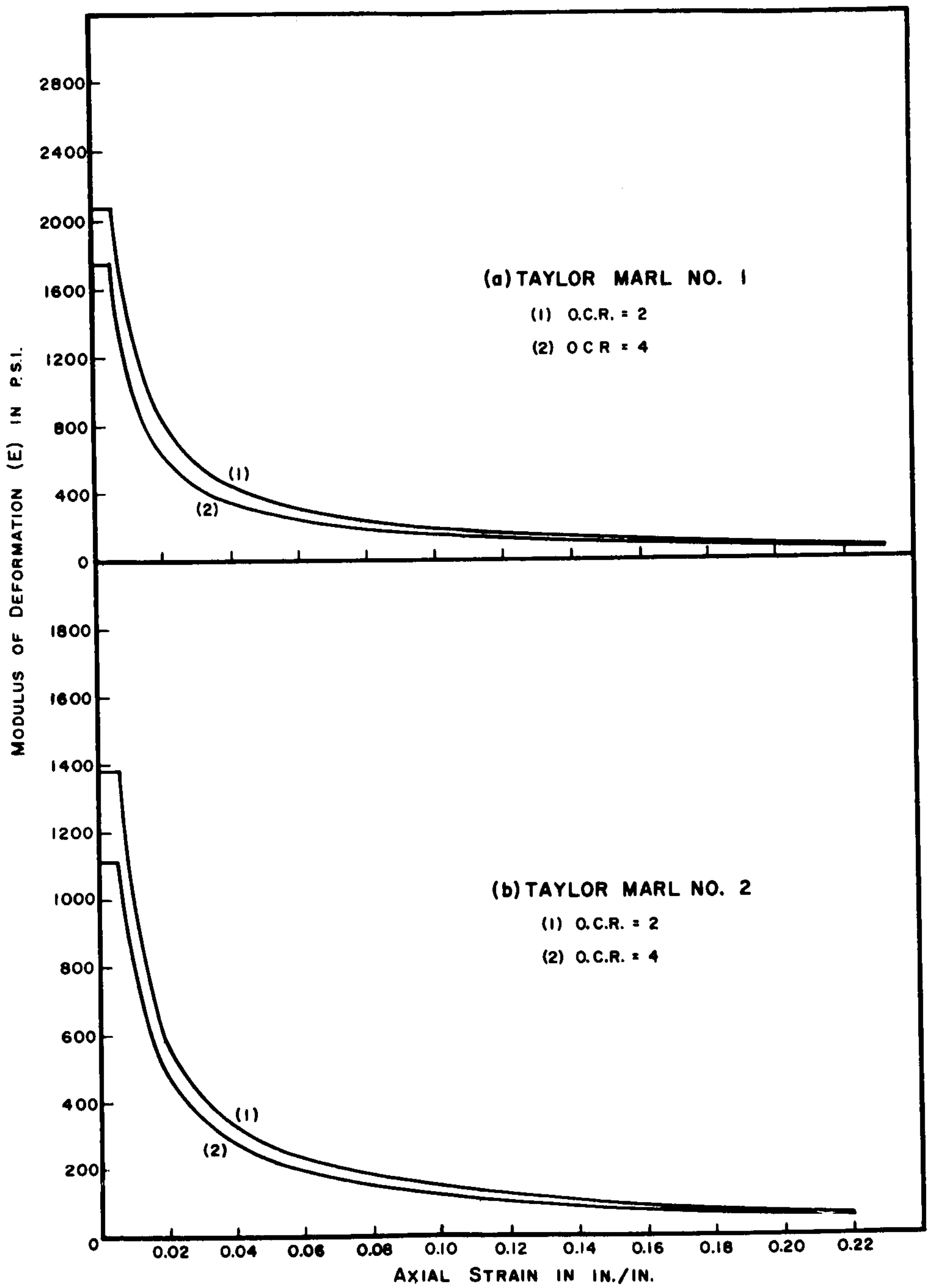


FIG. 26. MODULUS OF DEFORMATION VS. AXIAL STRAIN FOR OVER-CONSOLIDATED CLAYS (σ_0 = 40 p.s.i.)

with the factor of safety is given in Fig 27. The modulus in all cases remained constant to beyond half the maximum deviator stress and then decreased. The modulus of deformation, at any strain and for the same clay, increased with the decrease in the over-consolidation ratio. This was also the case for the initial modulus, and the modulus at any factor of safety.

Tables 23 and 24 show that the non-dimensional ratio E/σ_0 generally decreased with increased O.C.R., while E/σ_c and E/c increased (at any level of stress).

The increase in the plasticity index of the clay generally caused a decrease in the modulus and in the ratios E/σ_0 , E/c , and E/σ_c (at any level of stress and O.C.R.). The clay with the lower plasticity index also gave a higher modulus at any strain and O.C.R.

When compared with the results of normally-consolidated clays (with the same σ_c) presented in Art. 6.5, the modulus of deformation and the ratio E/σ_c of over-consolidated samples was found to be larger, while the ratio E/c was smaller.

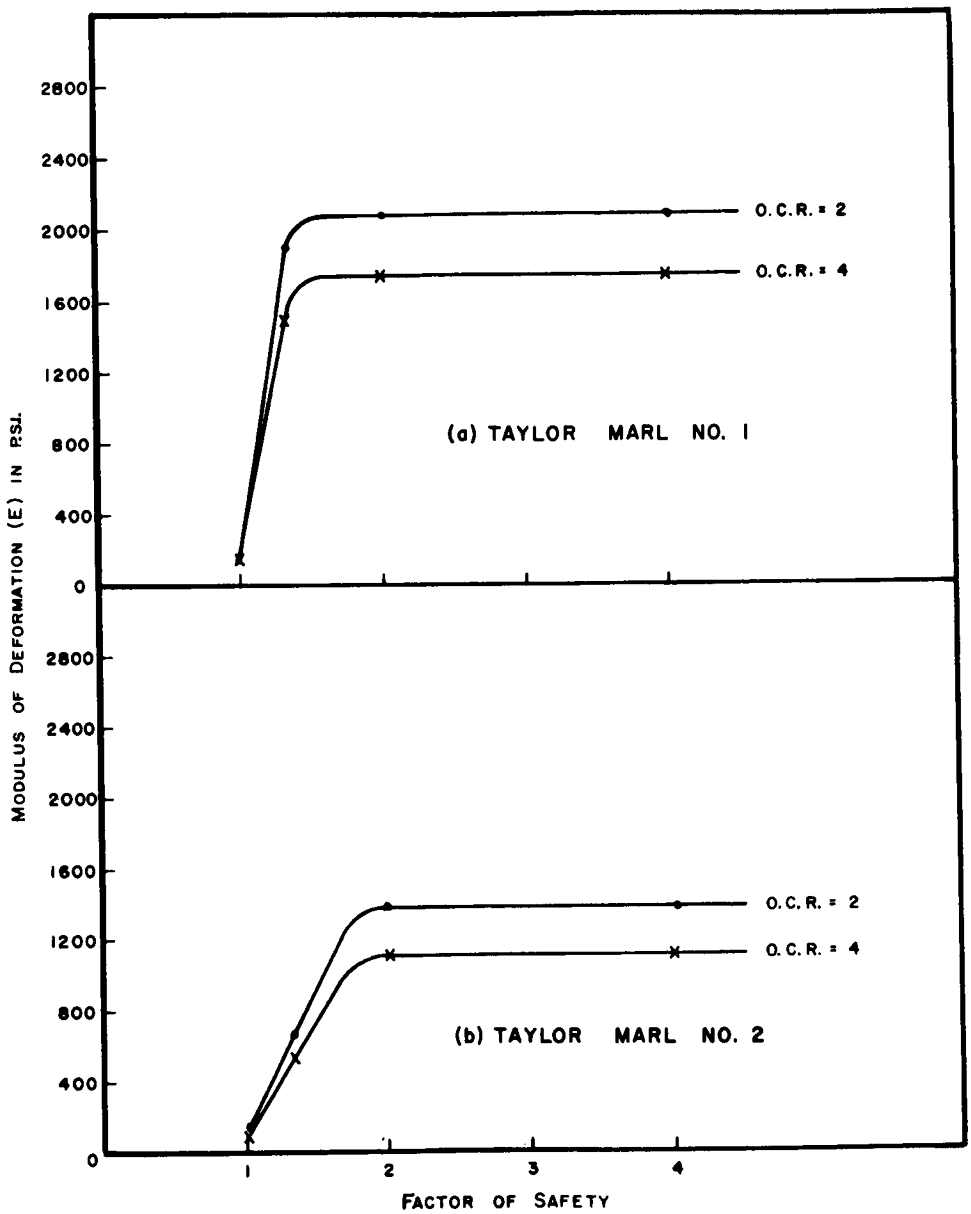


FIG. 27. MODULUS OF DEFORMATION VS. FACTOR OF SAFETY FOR OVER-CONSOLIDATED CLAYS (σ_3 = 40 p.s.i.)

CHAPTER VIII

UNCONFINED COMPRESSION TESTS ON CLAYS

8.1 General

A very limited number of unconfined tests were performed on clay samples prepared by vacuum extrusion. In these tests measurements of lateral deformation were made to evaluate the lateral strain ratio of clays.

The main object of these tests was to determine, in a very preliminary way, the value of μ for unconfined clay specimens. Also to see if there is any evidence of variation in the value of μ with changes in the level of strain and stress, moisture content, and plasticity of the clay.

The unconfined compression test was chosen in this case because of the difficulty of measuring lateral deformations of test specimens in the conventional triaxial compression test. Nevertheless the importance of evaluating the effect of confining pressures on the value of μ must be emphasized.

The clay specimens were tested as extruded, that is, without any consolidation whatsoever. This was done because of limitations in time. The clay samples in such case are believed to be slightly under-consolidated. The need for testing normally- and over-consolidated clay specimens must be pointed out.

Beside investigating the lateral strain ratio, an analysis of volume changes, occurring during the unconfined compression tests, is given in Appendix F. Also the axial stress-strain behavior of the unconfined clay specimens will be presented.

8.2 Sample Preparation

Specimens of Taylor Marl No. 1 and Vicksburg silty clay were extruded using the technique described in Art. 6.3. The moisture content of the Taylor Marl specimens was about 38% for the first group and 31% for the second group of specimens. The Vicksburg silty clay samples had about 28% moisture content. The reason for extruding Taylor Marl specimens with different moisture contents was to investigate the effect of varying moisture content on the lateral strain ratio of the clay. The effect of the plasticity of the clay was also determined by comparing the Vicksburg silty clay specimens with Taylor Marl No. 1, at about 31% moisture content.

The extruded clay samples were 2.8 in. diameter. When a sample was taken out of the moist room to be tested, its ends were trimmed to give a length equal to 5.6 in. The sample was then transferred to the machine as will be described in the next article.

8.3 Test Equipment and Procedure

The test equipment and set-up were similar to the ones desceibed in Art. 4.4. The same triaxial cell, with the lucite pressure chamber removed, was used. Dial extensometers are clamped in the same manner, to measure lateral deformations. The 2.8 in. diameter, 5.6 in. length clay specimen was placed on the base of the cell, which had a smooth circular 2.8 in. diameter aluminum plate on. Another similar smooth plate was placed on top of the sample to minimize friction at the ends.

The test procedure is similar to the one described in Art. 4.5. The speed of testing was the same for all tests, and was adjusted to give a rate of strain equal to 1.5 per cent per minute. The lower drainage valve of the cell was kept closed during the quick unconfined test and no rubber membrane was placed around the sample.

8.4 Results and Discussion

Computation of stresses in this case, as in the case of vacuum tests on sand, are based on areas of the specimens determined by actually measuring the lateral deformations during the test. It was observed that using smooth end plates reduced end friction considerably. The barreling effect was eliminated to a large extent in these tests. Therefore it was assumed that the average lateral deformation of the specimen at any time during the test, was equal to the deformation at its mid-height determined by the extensometer readings. It was also believed that by averaging the values of two perpendicular diameters at the mid-height of the specimen, one can assume the cylindrical shape to be retained throughout the test. In the present series of tests, it was noticed that perfect cylindrical shape of the test specimen was not kept all the time, but was better than in the case of sand. Except for the assumptions given above, the method of computation used was similar to the one presented in Art. 4.6.

A. Axial Stress-Deformation Properties

The results of the various tests are summarized in Table 25 and Fig 28, and the stress-strain curves are given in Appendix F. Comparing the results of Taylor Marl specimens, it can be seen that for the same clay the strain at any level of stress increased with the increase in moisture content. Also the stress and the modulus of deformation, at any strain or stress level, increased as the moisture content decreased, while the ratio E/c decreased.

B. Lateral Strain Ratio

Figure 28(b) indicates the variation of the lateral strain ratio with the level of strain. The value of μ increased with the increase in axial strain. This increase however became much smaller in the higher strain levels.

TABLE 25

UNCONFINED COMPRESSION TESTS ON CLAYS

(a) Axial Stress-Deformation Properties

Level of Stress	TAYLOR MARL NO.1, w=38.61%				TAYLOR MARL NO.1, w=31.23%				VICKSBURG SILTY CLAY			
	ϵ (in/in)	σ_Δ (psi)	E (psi)	$\frac{E}{c}$	ϵ (in/in)	σ_Δ (psi)	E (psi)	$\frac{E}{c}$	ϵ (in/in)	σ_Δ (psi)	E (psi)	$\frac{E}{c}$
At the end of the straight-line portion	0.0008	0.498	623	436	0.0040	4.510	1128	290	0.0028	1.650	589	305
At half the maximum stress	0.0120	1.430	119	83	0.0035	3.890	1128	290	0.0085	1.930	227	118
At maximum deviator stress	0.1082	2.860	26	18	0.1002	7.780	78	20	0.2000	3.860*	19	100

(b) Lateral Strain Ratio

Level of Stress	Taylor Marl No.1,w=38.61%	Taylor Marl No.1,w=31.23%	Vicksburg Silty Clay
At the end of the straight-line portion	0.342	0.387	0.317
At half the maximum stress	0.403	0.385	0.347
At maximum deviator stress	0.479	0.491	0.478*

*Extrapolated values

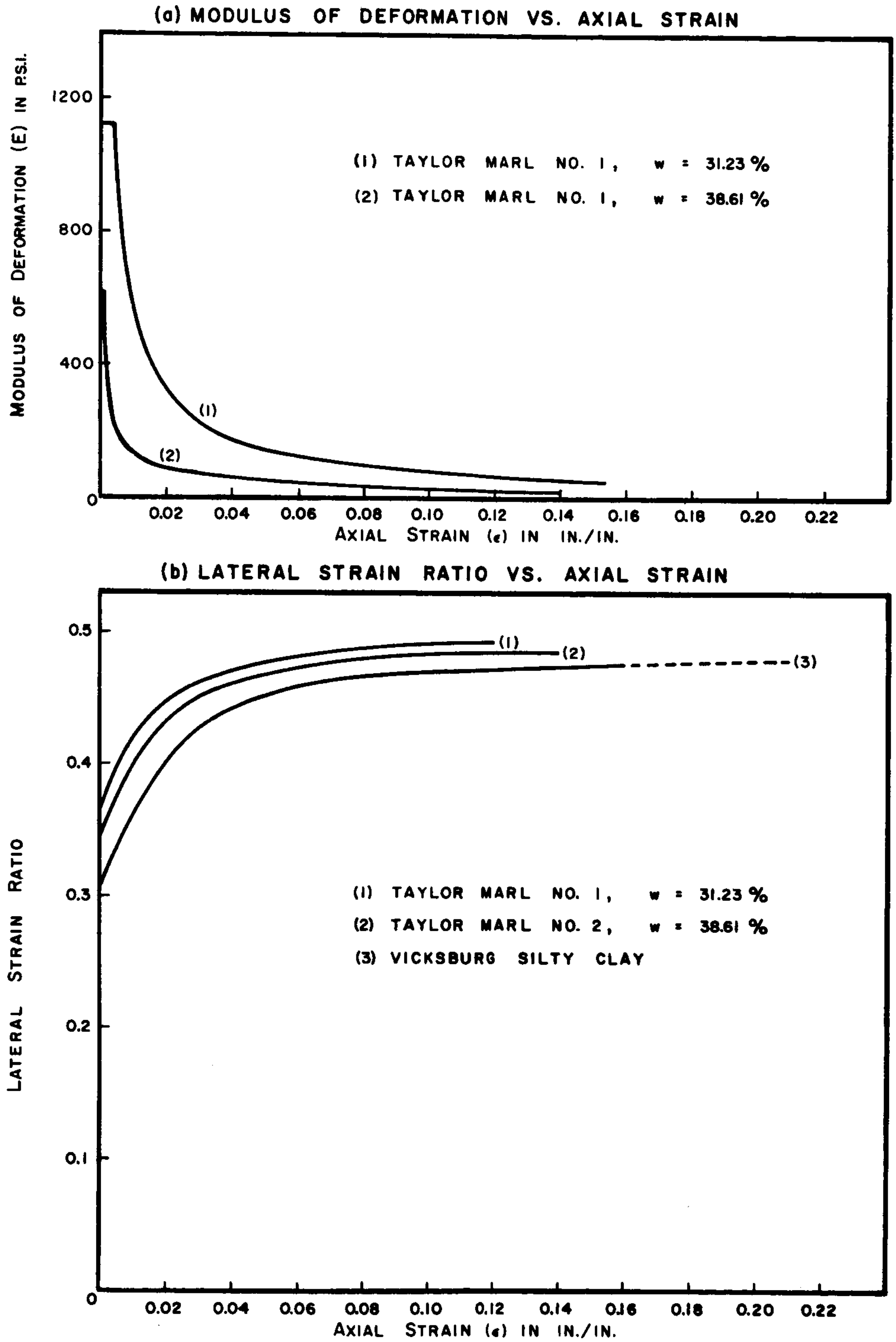


FIG. 28. UNCONFINED COMPRESSION TESTS ON CLAYS

Table 25(b) shows the effect of the level of stress on the value of μ . As the stress level increased, the lateral strain ratio increased in all tests.

The presented results indicate that the moisture content and the plasticity of the clay has some effect on the lateral strain ratio. The value of μ , at any level of stress or strain, was larger for the lower moisture content and higher plasticity index. The effect of these two variables was small. More tests are required before final conclusions are made.

CHAPTER IX

CONCLUSIONS

The following conclusions are drawn from the findings of this study:

1. The stress-strain properties of soils can be described by two quantities, the modulus of deformation and lateral strain ratio, if the pattern of variation of the two parameters can be determined.

2. The stress-strain behavior of sand was influenced by the density, confining pressure, rate of strain, and level of stress and strain. The properties of the clay were affected by the consolidation pressure, moisture content, confining pressure, plasticity index, over-consolidation ratio, and level of stress and strain.

3. The triaxial test, in spite of its limitations, is considered to be the best available test at the present time for studying the laboratory stress-strain behavior and strength characteristics of soils. This evaluation is based on a literature survey conducted by the author.

4. The modulus of deformation of the soil determined by quick triaxial tests, was constant for only a small value of strain, after which it decreased with the increase in strain. The modulus was also constant to a certain level of stress, and then decreased as the stress increased.

5. The lateral strain ratio of the soil increased with the increase in the level of stress or strain. This increase was more for dense than for loose sand. For clay and loose sand the increase was much less at higher strains.

6. The modulus of deformation of sand, at any level of stress or strain,

increased with the increase in density or confining pressure. The effect of the limited range of rates of strain investigated on the modulus of sand was not pronounced. Only a small increase in the modulus was generally observed when higher rates of strain were used.

7. The lateral strain ratio for sand, at the same level of stress or strain, increased as the density increased (other factors being constant). This ratio varied as the confining pressure was changed, but no regular pattern of variation could be detected, for the small range of pressures used. The rate of strain did not affect the lateral strain ratio of sand to any appreciable degree. A small decrease in this ratio sometimes occurred with the limited increase in the rate of strain experienced in this study.

8. The modulus of deformation of normally-consolidated clays, at any level of strain or stress, increased with the increase in the consolidation pressure (or the decrease in moisture content) and decreased as the plasticity index of the clay increased. The modulus also decreased as the confining pressure in the triaxial test decreased, for clays having the same consolidation pressure.

9. The modulus of deformation of over-consolidated clay, at any level of stress or strain, was higher than the modulus for the same clay normally-consolidated under the same consolidation pressure. For over-consolidated clay, with the same over-consolidation pressure, the modulus of deformation decreased as the over-consolidation ratio increased (that is as the consolidation pressure decreased and moisture content increased), other factors being constant. This modulus also decreased with the increase in plasticity index.

10. The lateral strain ratio for unconfined clay was affected by the variation in moisture content and plasticity of the clay. Based on the results

of a very small number of tests on vacuum extruded clay specimens, it was found that this ratio, at any level of stress or strain, generally increased with the decrease in moisture content and/or increase in plasticity index of the clay.

11. The angle of internal friction of sand increased with the increase in density and rate of strain. The effect of the rate of strain, though small, was more for dense than loose sand. Any rate of strain, within the investigated range, may be used in the laboratory determination of the strength and stress-strain characteristics of sand with no major variation in the results to be expected.

12. A correlation exists among the moisture content, cohesion, and plasticity of normally consolidated clays. This relation is helpful in predicting the strength of the undisturbed clay from the results of simple tests on disturbed samples.

13. The deviator stress at any level of stress or strain increased, the initial linear portion of the stress-strain curve and its slope (that is, the stress and strain at which the curve is no longer a straight line) increased, the ratio $\sigma_{\Delta_1}/\sigma_{\Delta_{max}}$ increased, the ratio $\sigma_{\Delta_{0.1}}/\sigma_{\Delta_{max}}$ or $\sigma_{\Delta_{0.2}}/\sigma_{\Delta_{max}}$ decreased, and the axial strain at maximum deviator stress decreased with the increase in density, confining pressure and rate of strain in the case of sand. The same variation occurred in the case of normally-consolidated clays as the consolidation or confining pressure increased, and moisture content or plasticity index decreased. This pattern also developed when the same clay was over-consolidated and had the same consolidation pressure as the normally consolidated clay, also for over-consolidated clay (with the same over-consolidation pressure) as the O.C.R. decreased.

14. The non-dimensional ratios E/σ_3 and $E/(\sigma_3 \tan \phi)$ for sand, and E/σ_c and E/c for clay were found to vary (within narrow limits, at any level of stress or strain) with the density confining pressure, rate of strain, consolidation pressure, moisture content, plasticity index, and over-consolidation ratio. The pattern of variation of these ratios with the different variables was given in this study. Average values for these ratios can be determined, however, for a limited range of variation in the various factors.

15. The accumulation of information and values of the stress-strain properties for various soils, such as those presented in chapters 4 through 8, can be used to predict the behavior of a soil when no sample is available.

16. Volume changes during shear in vacuum triaxial test specimens on dry sand, and unconfined clay specimens, were determined by measuring axial and lateral deformations of the sample using extensometers. There was an indication that volume changes for sand may be influenced by the confining pressure and rate of strain. Dense sand increased in volume during the test, while loose sand decreased in volume. The volume decrease that took place in the case of a limited number of unconfined tests on clay was affected by the moisture content and plasticity index of the clay.

17. The assumptions used in computing the results of conventional triaxial tests (no volume change and perfect cylindrical shape during the test) were not found to be absolutely true. However, the results were very close to the actual values, particularly in the lower range of strains. Therefore the use of such assumptions is justified to a certain extent.

CHAPTER X

RECOMMENDATIONS FOR FUTURE RESEARCH

The present investigation revealed several points that are worthy of further pursuit and study, among which are:

1. Development of new testing equipment and procedures for laboratory studies of stress-deformation characteristics of soils under various conditions. Such methods must reproduce, as closely as possible, all natural conditions encountered in the field and must not include any of the shortcomings of present day facilities. Examples of the suggested equipment are a plane strain apparatus or triaxial equipment with separate control of all principal stresses.

2. Modification of the triaxial test, and other available equipment, to eliminate its disadvantages. The main problem in triaxial equipment are end restraint due to friction and the difficulty in measuring lateral deformations of the test specimen.

3. Investigation of the various factors believed to influence the stress-strain properties of soils, for example the effect of the different types of loading on stresses and strains. Such studies must also include a determination of the modulus of deformation and lateral strain ratio for soils under various conditions. The importance of investigating the effects of confining pressures on the lateral strain ratio, and higher rates of strain on the stress-strain properties in general, must be emphasized.

4. Accumulation of laboratory stress-strain data for many soils under different conditions, following the steps outlined in the present study. Such

information must then be analyzed statistically to yield the ranges within which the stress-strain properties of various soil categories may be expected to lie. This will help in describing the stress-strain behavior of a soil, under a certain set of conditions, when no undisturbed sample is available.

5. Investigation of the correlation suggested among moisture content, cohesion, and soil plasticity for normally consolidated clays. Such a correlation should be based on the results of a great number of tests and can be used to predict the undisturbed strength of normally-consolidated clays from simple tests on disturbed samples. The possibility of other correlations relating various soil properties together must also be checked.

6. Correlation of load-settlement data, from full-scale field tests on various foundation elements, with laboratory stress-strain properties of soils. Such correlation will enable the prediction of field behavior from data determined in the laboratory. It will also eliminate any complaint of impericism of the laboratory data and is vitally important until a laboratory test is developed that will fully reproduce all field conditions, a goal which in the author's opinion must be acknowledged impossible.

APPENDICES

APPENDIX A

STRESS-STRAIN CURVES OF SANDS

i. Seating Error

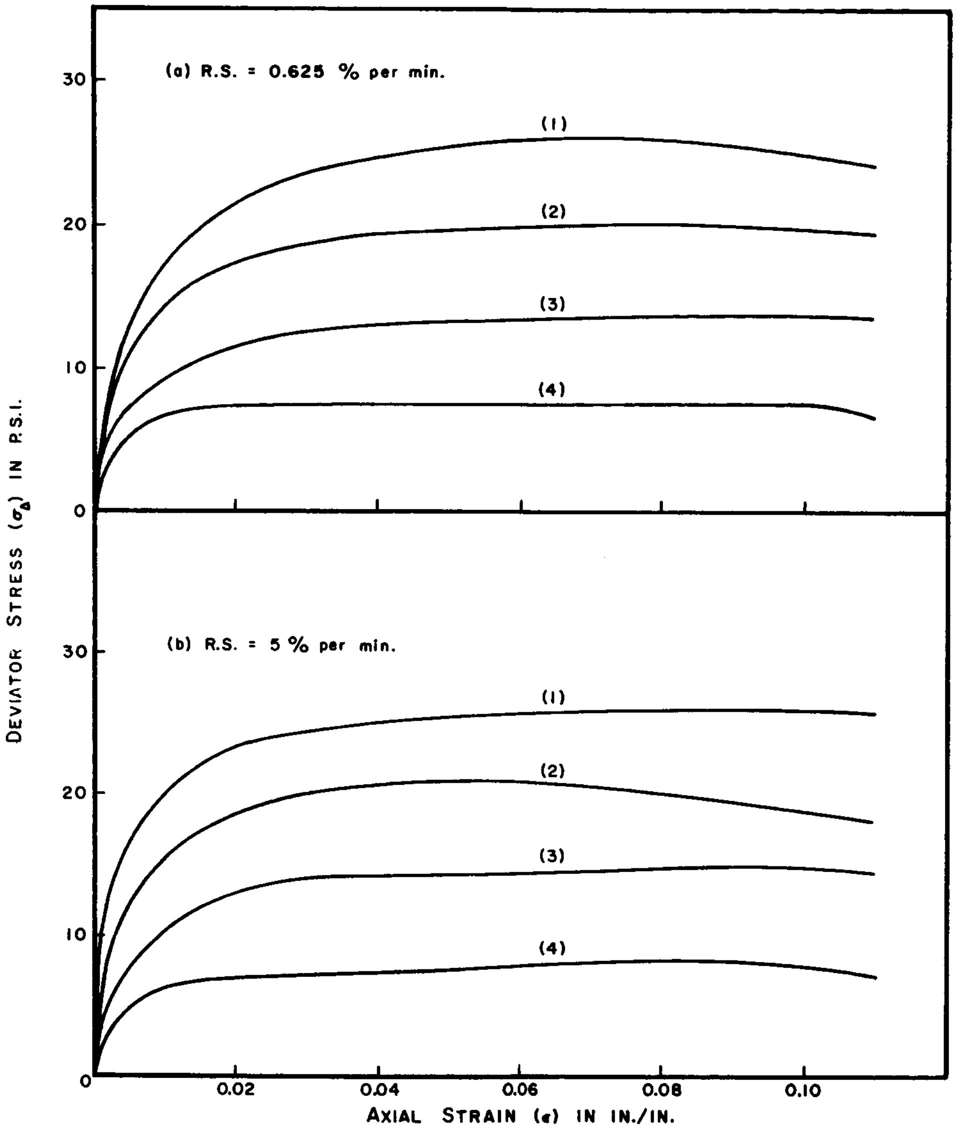
One of the main difficulties encountered in the determination of the stress-strain curves for soils, particularly sand, is the seating error. Most stress-strain curves of granular soils exhibit the initial non-linear portion to a greater or lesser degree. This may occur due to seating error, resulting from the uneven bearing surfaces at the ends of the specimen (since the ends of granular specimens can not be trimmed flush after preparation and must be hand-finished and smoothed.) Uneven distribution of stresses at the ends of the specimen during loading is thus produced.

The non-linear portion of stress-strain curves, due to seating error, appeared in some of the tests performed in the present study. In these cases a correction had to be introduced to overshadow some of the undesirable effects of the seating error.

The correction used consists of producing the initial straight line portion of the stress-strain curve to intersect the strain axis at a point. The origin of the curve is shifted to this intersection point. In this way only the initial portion of the curve is changed, by eliminating the non-linear part, while the remainder of the curve remains the same.

ii. Stress-Strain Curves

The stress strain curves that are presented here were obtained by drawing the deviator stress versus the axial strain. Figs 29, 30, 31, and



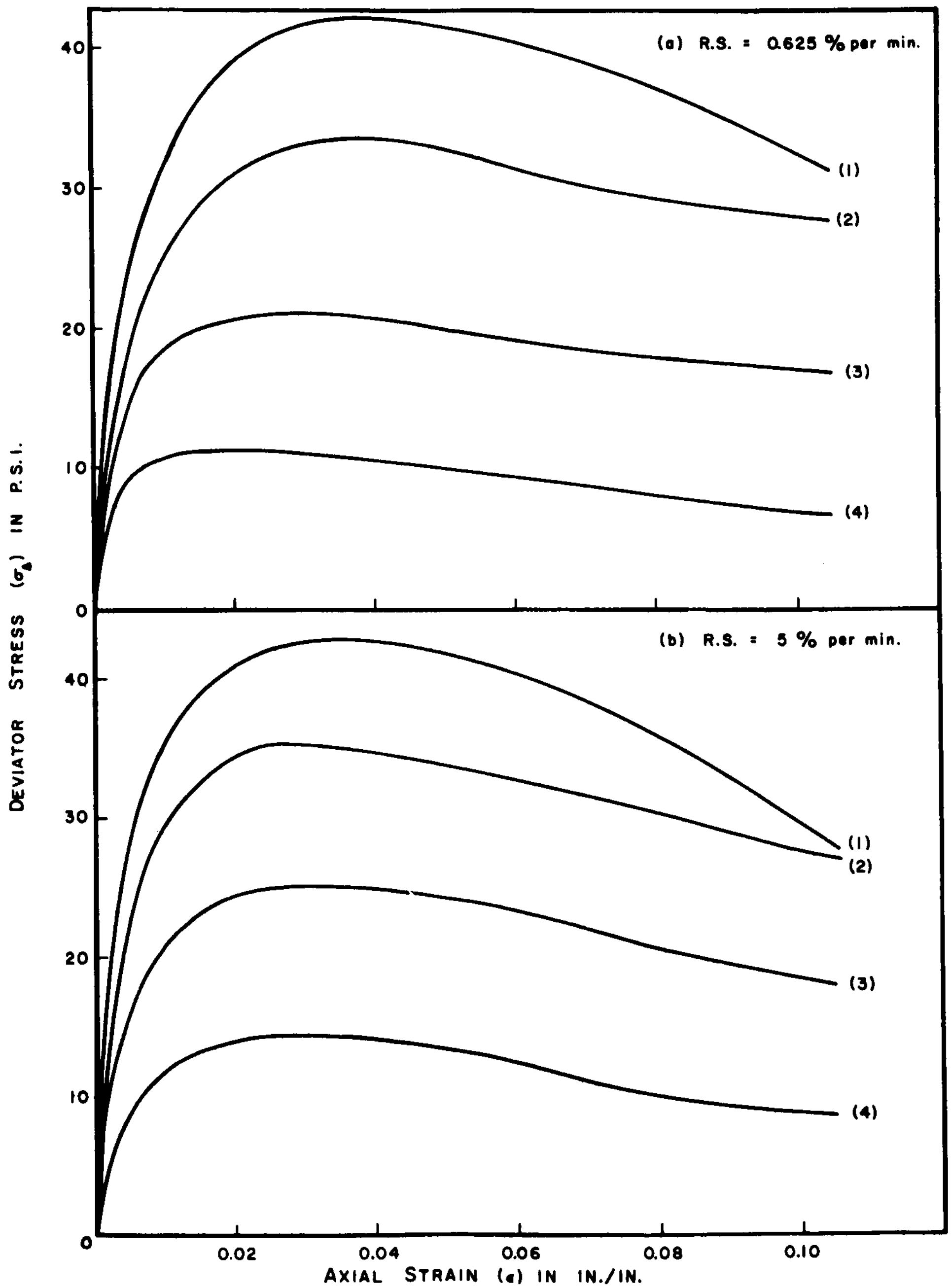
DRY COLORADO RIVER SAND

DENSITY = 94 p.c.f.

(1) σ_3 = 8.69 p.s.i. (2) σ_3 = 6.95 p.s.i.

(3) σ_3 = 4.64 p.s.i. (4) σ_3 = 2.32 p.s.i.

FIG. 29. STRESS-STRAIN CURVES



(1) σ_3 = 8.69 p.s.i. (2) σ_3 = 6.95 p.s.i. (3) σ_3 = 4.64 p.s.i. (4) σ_3 = 2.32 p.s.i.

DENSITY = 102 p.c.f.

FIG. 30. STRESS VS. STRAIN CURVES FOR DRY COLORADO RIVER SAND

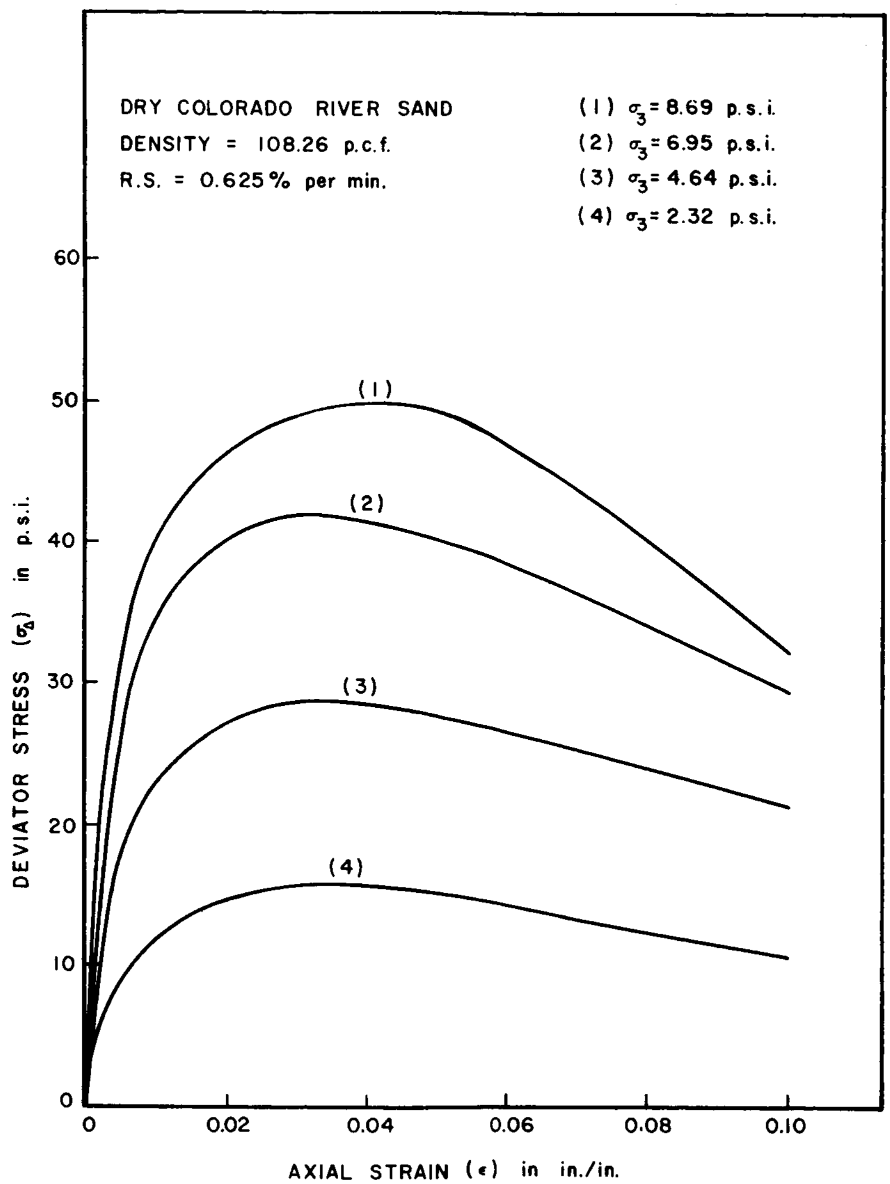


FIG. 31. STRESS-STRAIN CURVES
OF DRY COLORADO RIVER SAND

32 show the axial stress-strain curves for dry Colorado River sand at various conditions of density, confining pressure, and rate of strain. The stress-strain curves for dry Ottawa sand with two densities and confining pressures are given in Fig 33.

The deviator stresses in the case of Colorado River sand were computed using areas determined from the measured deformations. In the case of Ottawa sand, however, the stress was computed in the conventional way, assuming no volume change.

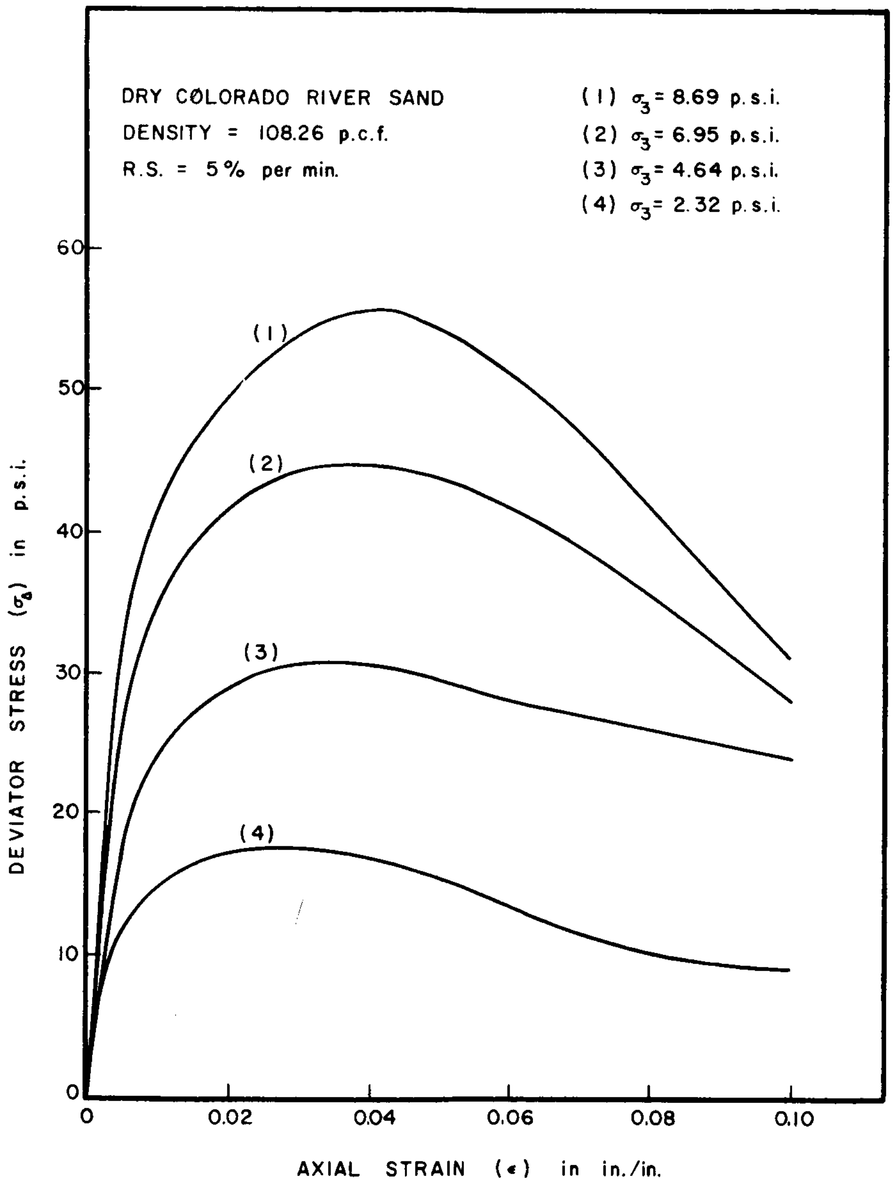


FIG. 32. STRESS-STRAIN CURVES OF DRY COLORADO RIVER SAND

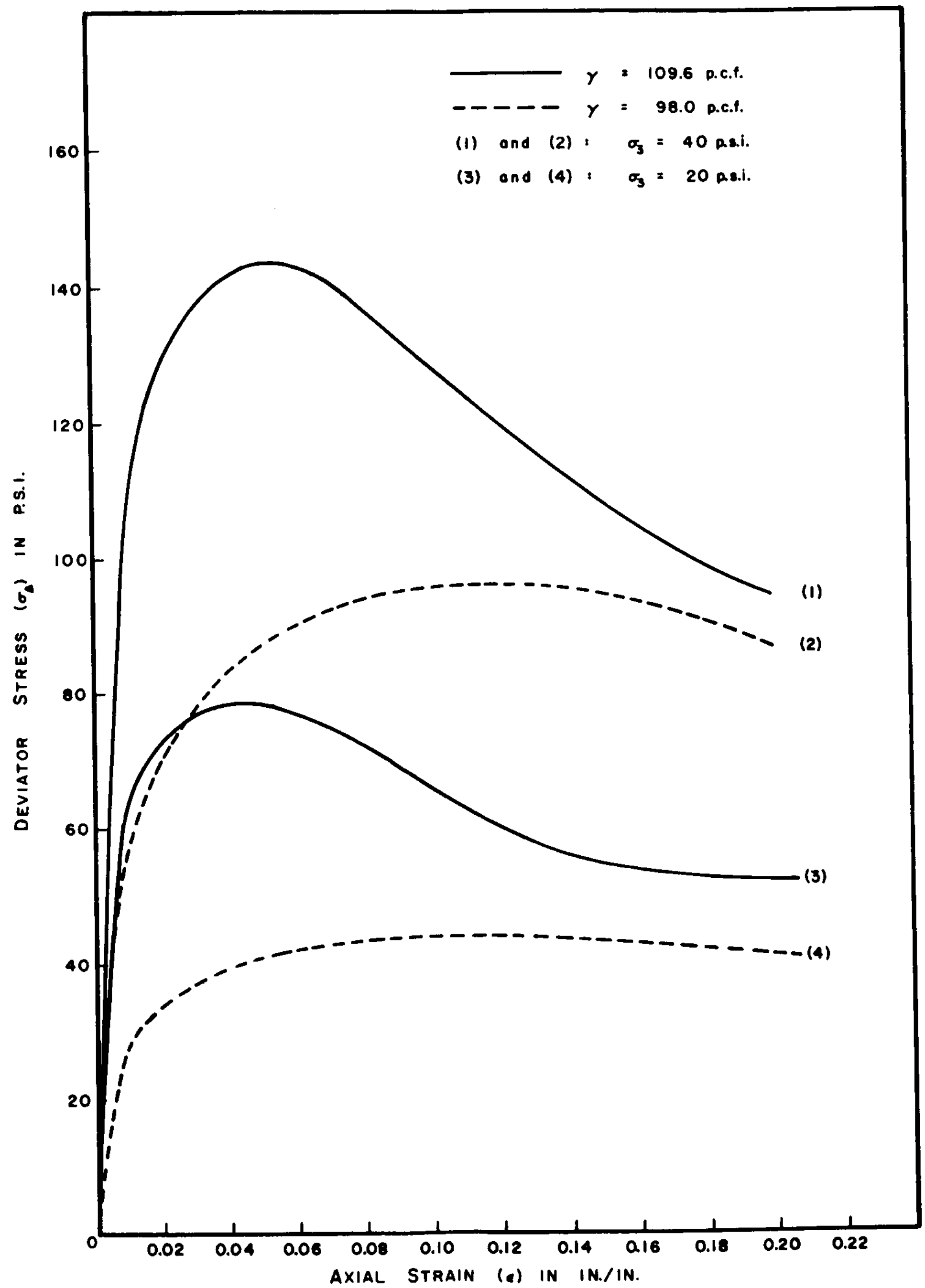


FIG. 33. STRESS-STRAIN CURVES FOR DRY OTTAWA SAND

APPENDIX B

MOHR CIRCLES AND ENVELOPES OF SANDS

(Angle of Internal Friction)

The angle of internal friction of the Colorado River sand was determined at three densities (108.26, 102, and 94 p.c.f.) and four rates of strains (0.625, 1.25, 2.5, and 5% per minute). For each density and rate of strain four vacuum triaxial tests were run (with σ_3 values of 2.32, 4.64, 6.95, and 8.69 psi). For each of these tests the maximum stress ordinate of the stress-strain curve was taken to be the maximum deviator stress $\sigma_{\Delta_{max}}$. Knowing the value of σ_3 and $\sigma_{\Delta_{max}}$ for each test a Mohr circle could be plotted. For each density and rate of strain four circles were thus determined. The straight line envelope, tangent to the four circles and passing through the origin was drawn to give the angle of internal friction for this condition.

The Mohr circles and envelopes, for the maximum and minimum rates of strain tested, are shown on Figs 34, 35, and 36. It can be noticed that the envelopes in some cases are not perfectly tangent to all four circles. This can be expected for the various reasons discussed in Art. 4.6. The results, however, are quite satisfactory.

Figure 37 shows the Mohr circles and envelopes for the dry Ottawa sand. These were determined in the same way as for the Colorado River sand. The angle of internal friction was greater for the higher density.

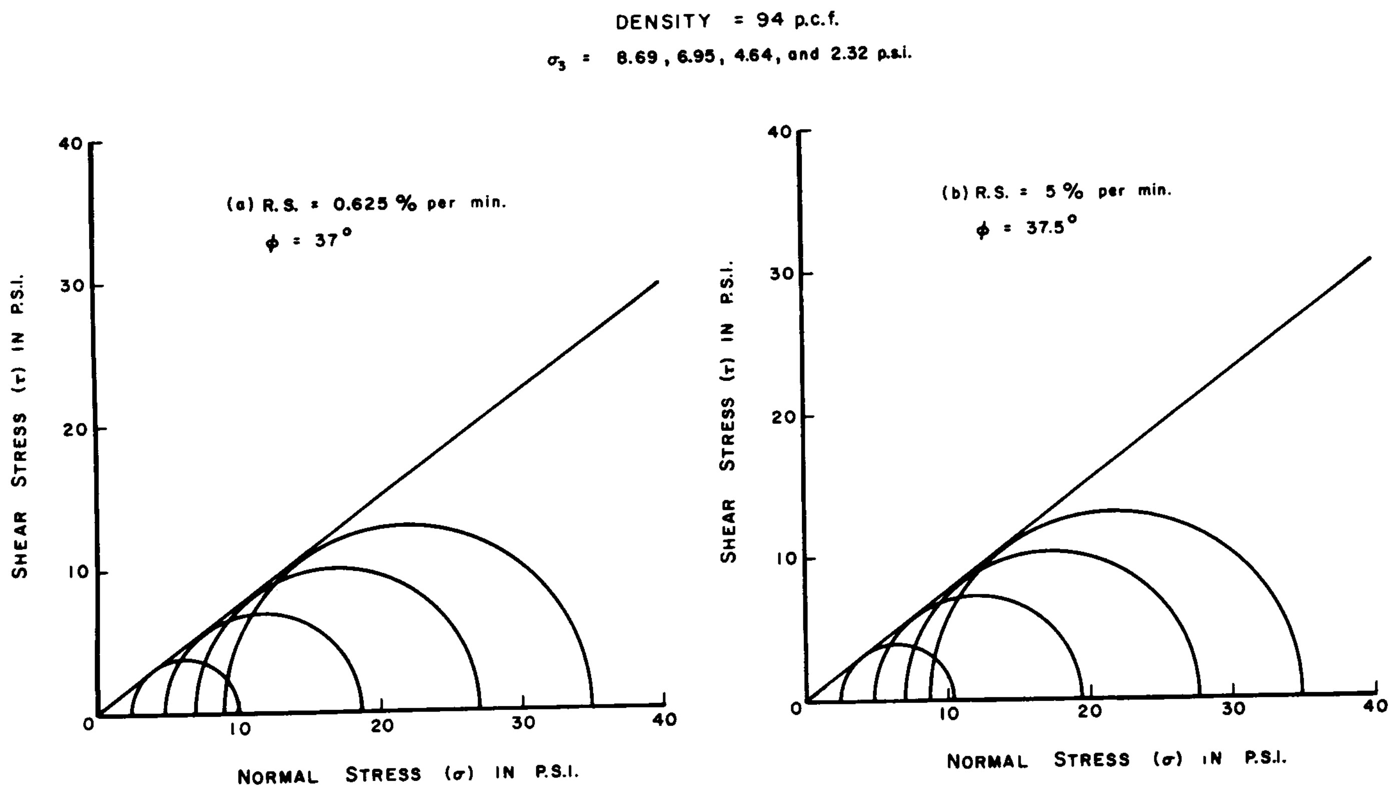
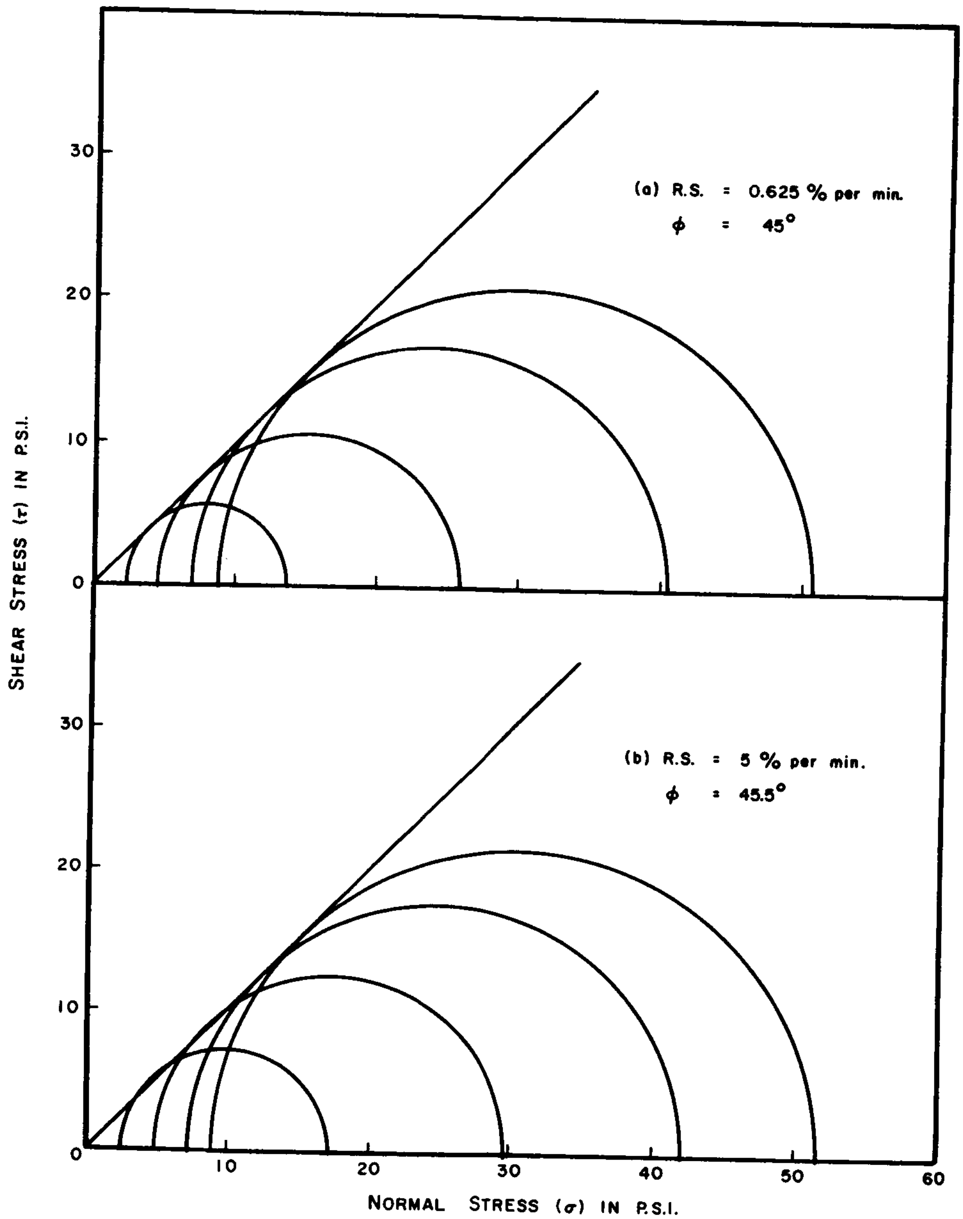


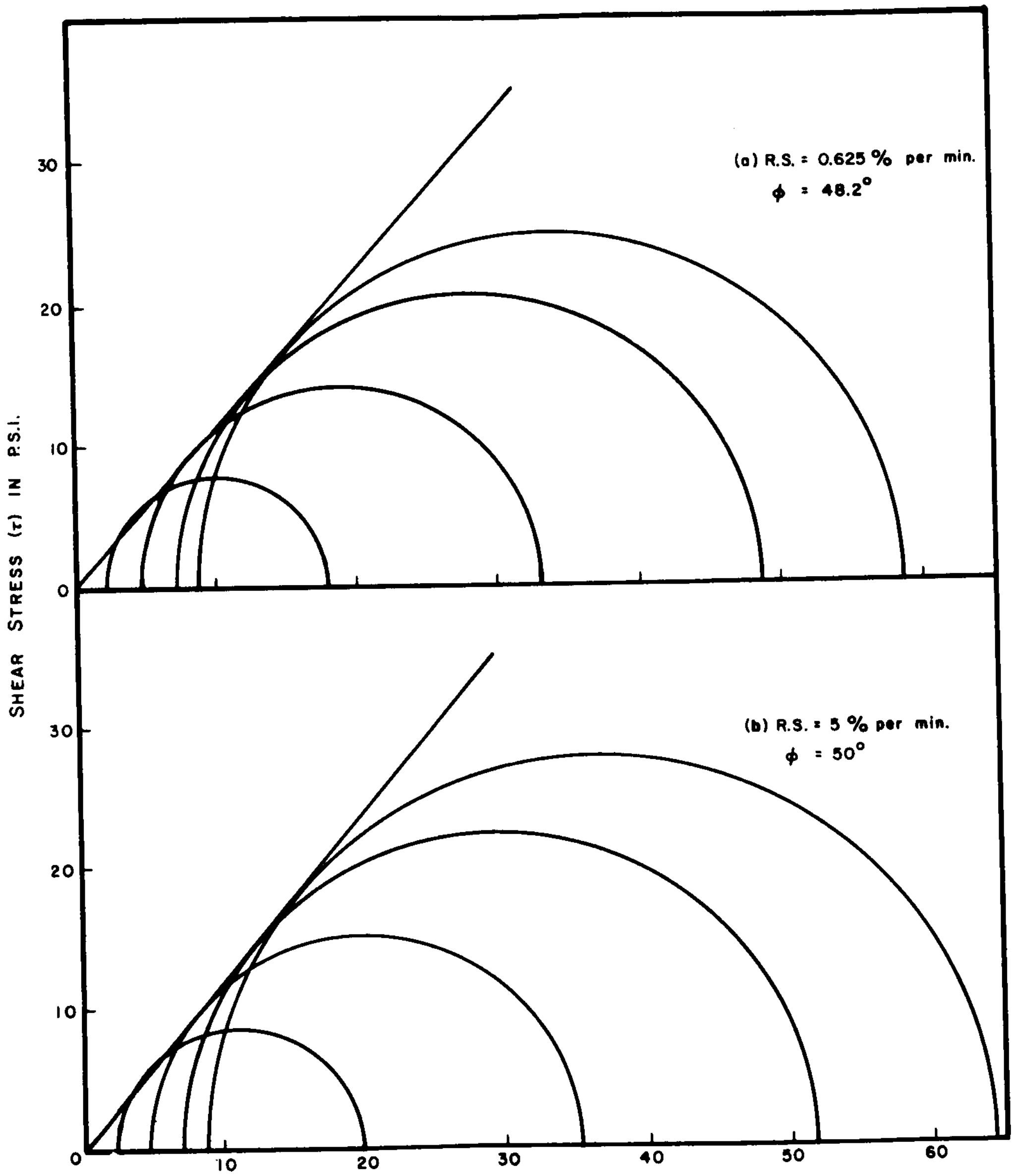
FIG. 34. MOHR DIAGRAMS FOR DRY COLORADO RIVER SAND



DENSITY = 102 p.c.f.

σ_3 = 8.69, 6.95, 4.64, and 2.32 p.s.i.

FIG. 35. MOHR DIAGRAM FOR DRY COLORADO RIVER SAND



DENSITY = 108.26 p.c.f.

σ_3 = 8.69, 6.95, 4.64, and 2.32 p.s.i.

FIG. 36. MOHR DIAGRAMS FOR DRY COLORADO RIVER SAND

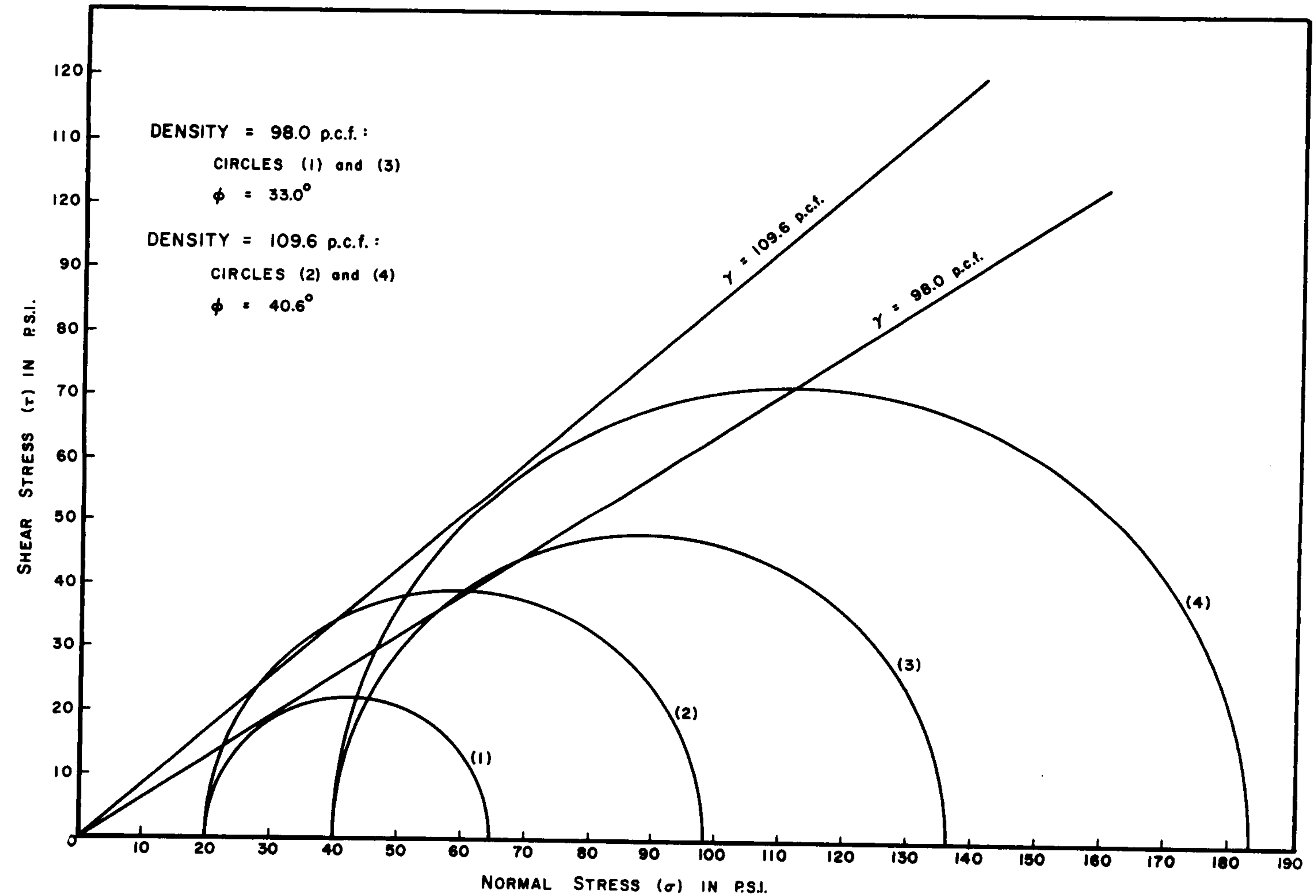


FIG. 37. MOHR CIRCLES AND ENVELOPES OF DRY OTTAWA SAND

APPENDIX C

STRESS-STRAIN CURVES FOR NORMALLY-CONSOLIDATED CLAYS

The stress-strain curves for the normally-consolidated clay specimens are shown in Figs 38 and 39. The curves for both Taylor Marl No. 1 and 2 peak to a maximum deviator stress and then the stress reduces to a smaller residual value. In the case of Vicksburg silty clay, the stress-strain curves do not show a maximum value, and the maximum deviator stress is chosen arbitrarily at a strain value of 0.2 in. per in.

The results of the triaxial tests were interpreted making the conventional assumptions of no volume change and perfect cylindrical shape of the specimen during a test. Each stress-strain curve shown in this study is the graphical average of the results of two identical tests.

Figure 40 shows the stress-strain curves for Taylor Marl No. 1 and Vicksburg silty clay specimens tested at a confining pressure other than the consolidation pressure.

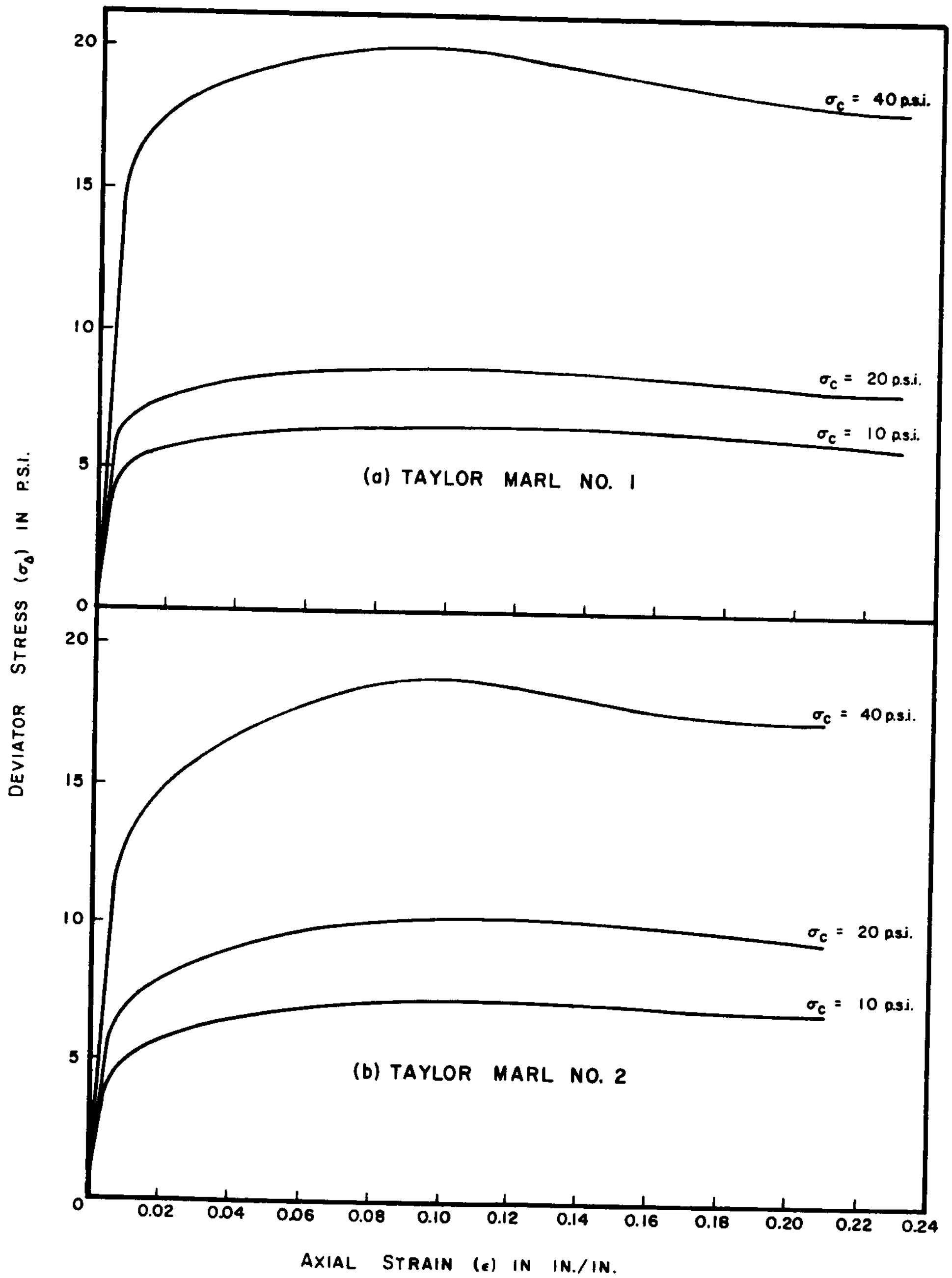


FIG. 38. STRESS-STRAIN CURVES FOR NORMALLY-CONSOLIDATED CLAY SPECIMENS

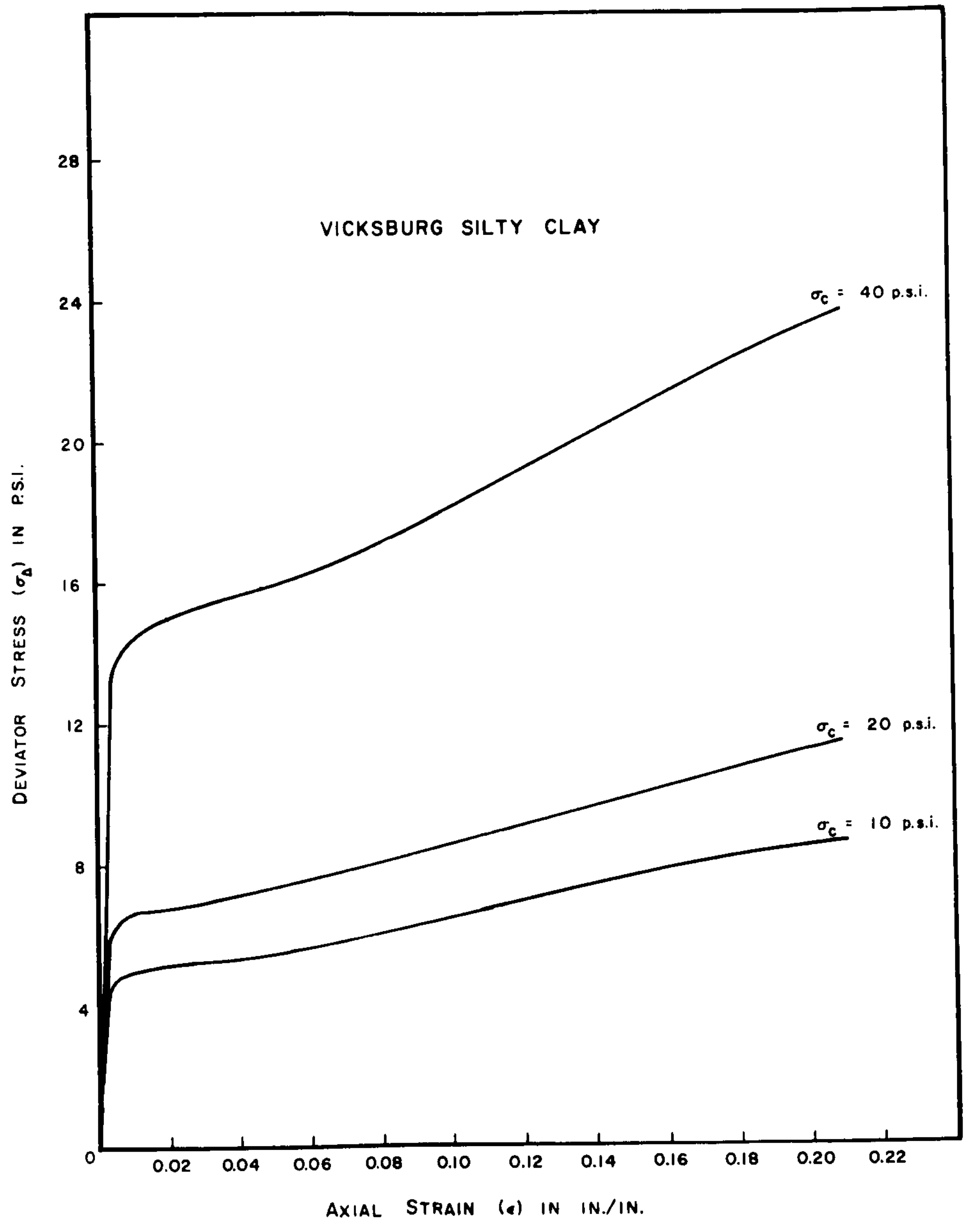
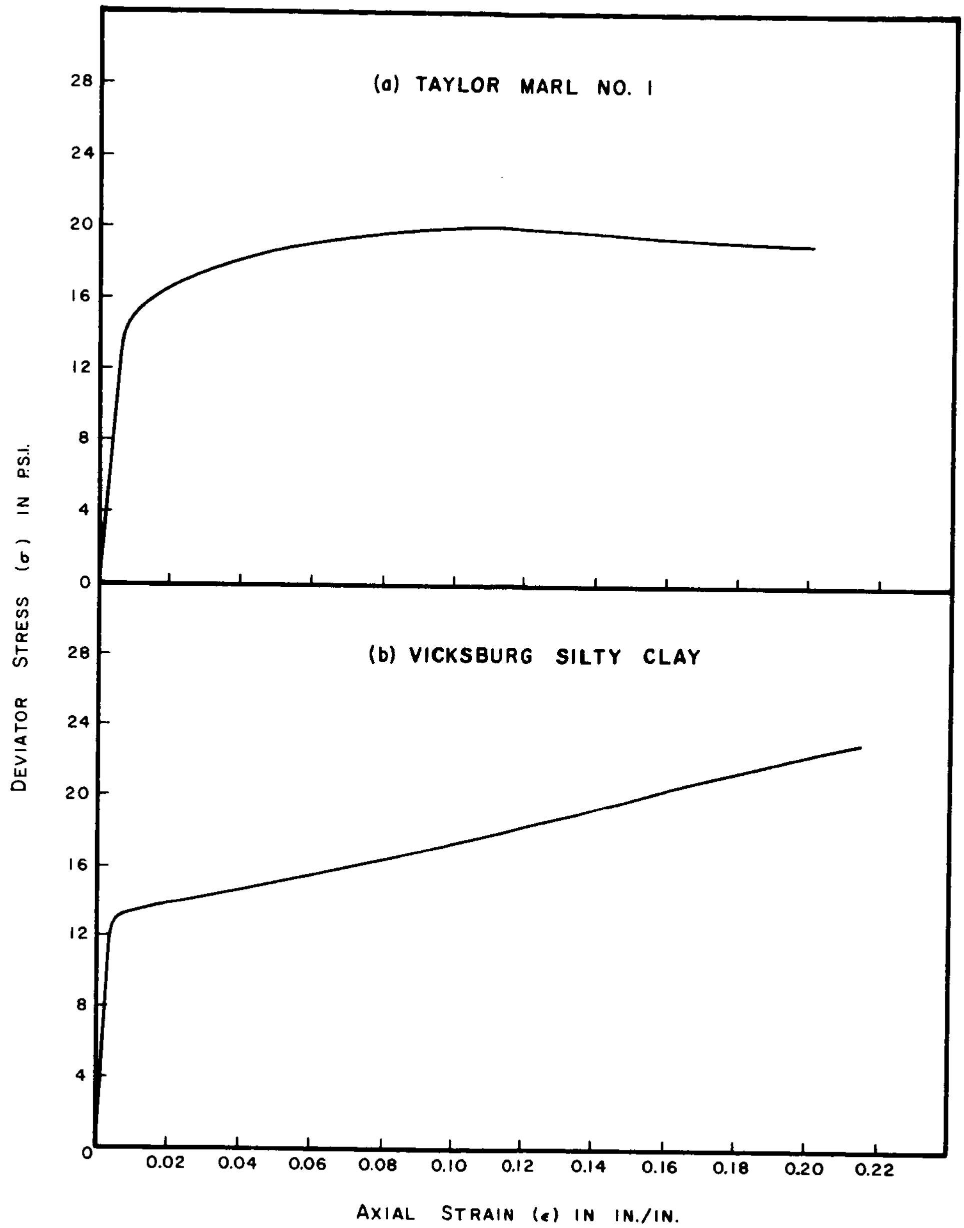


FIG. 39. STRESS-STRAIN CURVES FOR NORMALLY-CONSOLIDATED CLAY SPECIMENS



CONSOLIDATION PRESSURE (σ_c) = 40 p.s.i.
CONFINING PRESSURE (σ_3) = 20 p.s.i.

FIG. 40. STRESS-STRAIN CURVES FOR NORMALLY-CONSOLIDATED CLAY SPECIMENS TESTED AT OTHER THAN CONSOLIDATION PRESSURE

APPENDIX D

CORRELATION AMONG MOISTURE CONTENT, COHESION, AND PLASTICITY OF NORMALLY-CONSOLIDATED CLAYS

The suggested correlation is based on the results of a limited number of laboratory tests. Only three clays were investigated in this study. Specimens were extruded in the laboratory, then normally-consolidated in triaxial cells (isotropic consolidation) under three pressures, 10, 20, and 40 psi. The cohesion of each specimen was determined by running a quick (undrained) triaxial test. The cohesion is considered to be equal to half the maximum deviator stress, or the stress at 0.2 in. per in. strain if no maximum was apparent (based on the total stress concept). The moisture content of the sample was taken to be the average of the top, center, and bottom moistures, at the end of the triaxial test. The determination of moisture contents and Atterberg limits was carried out in accordance with the general accepted procedures used at The University of Texas (9).

Figure 41 gives the relation between the moisture content and cohesion (drawn on a log-scale) for the normally consolidated clay specimens. The results for each clay lie on a straight line, in this semi-logarithmic plot. The slope of each of these lines was drawn versus the plasticity index, then the liquid limit, of the clay (as shown in Fig 42). In either case the result was a straight line plot.

Because of insufficient data it cannot be concluded which of the two correlations is the one that generally holds better. But in any case the results show a definite trend and indicate the possibility of correlating

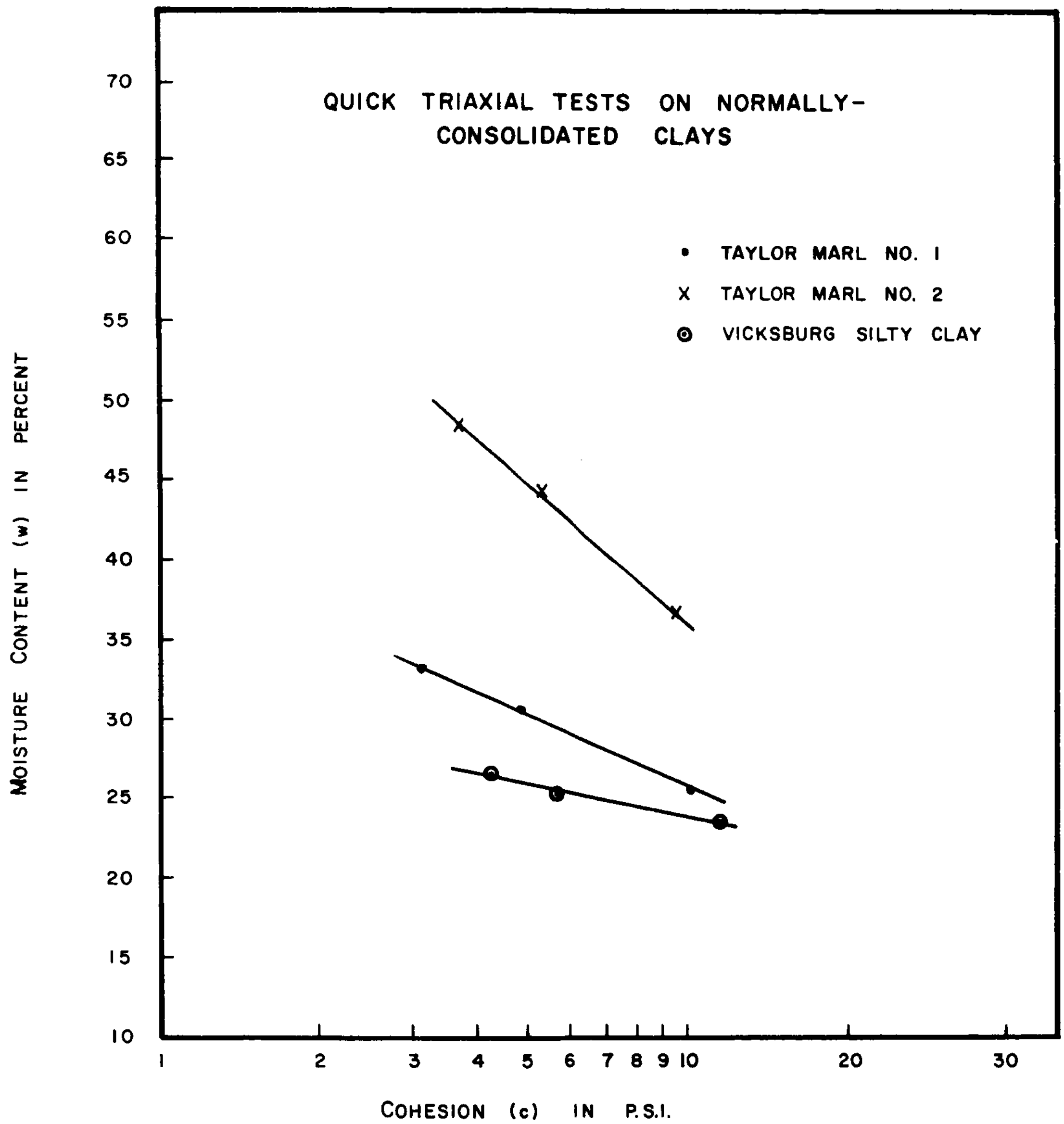


FIG. 41. PERCENT MOISTURE CONTENT VS. LOGARITM OF COHESION

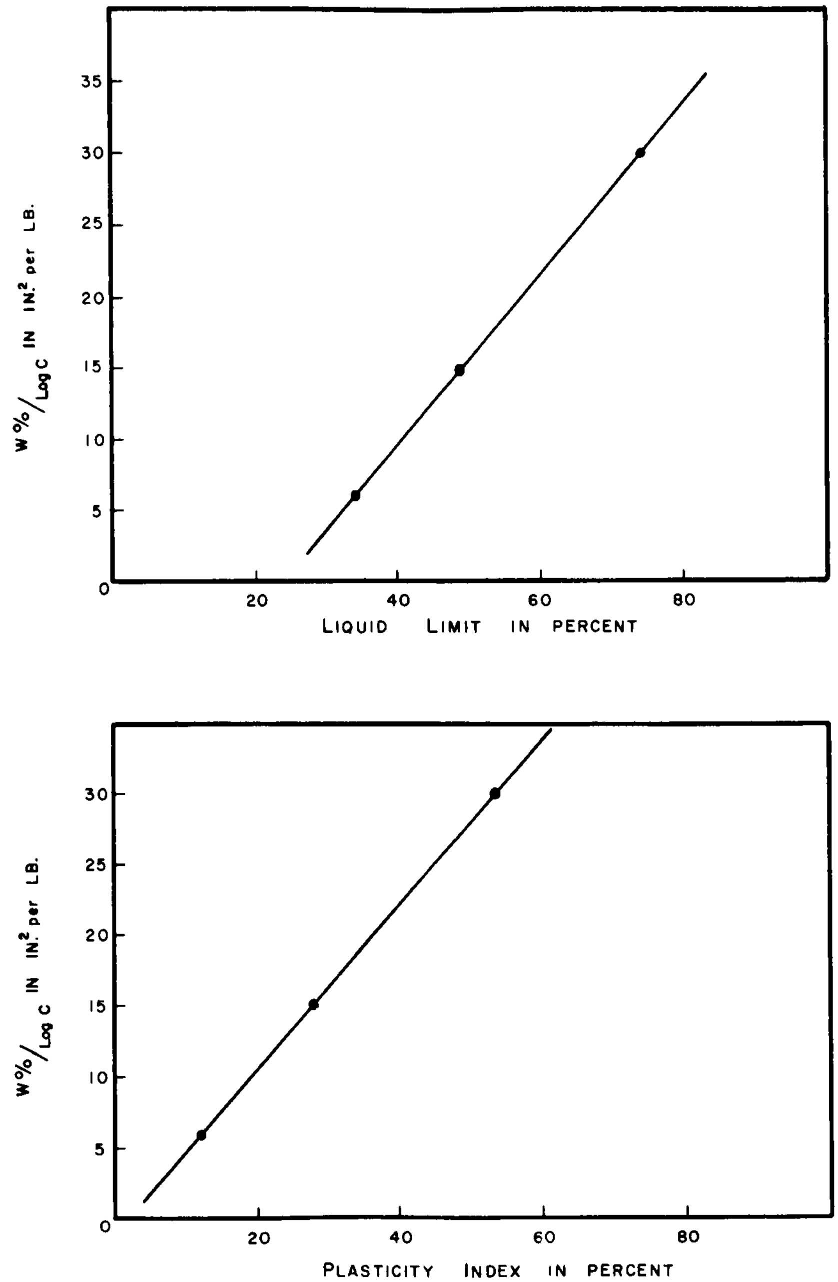


FIG. 42. CORRELATION AMONG MOISTURE CONTENT, COHESION, AND PLASTICITY OF NORMALLY-CONSOLIDATED CLAYS

moisture content, cohesion, and plasticity (either L.L., or P.I.) of normally-consolidated clays. More data is required before any final correlation is reached. The author suggests that this point be further investigated by studying undisturbed field samples of normally-consolidated clays. If a general correlation is reached, at least for clays in the same region and with the same origin, the cohesion of undisturbed clay may be determined by measuring the moisture content and Atterberg limits of a disturbed sample of the same clay.

APPENDIX E

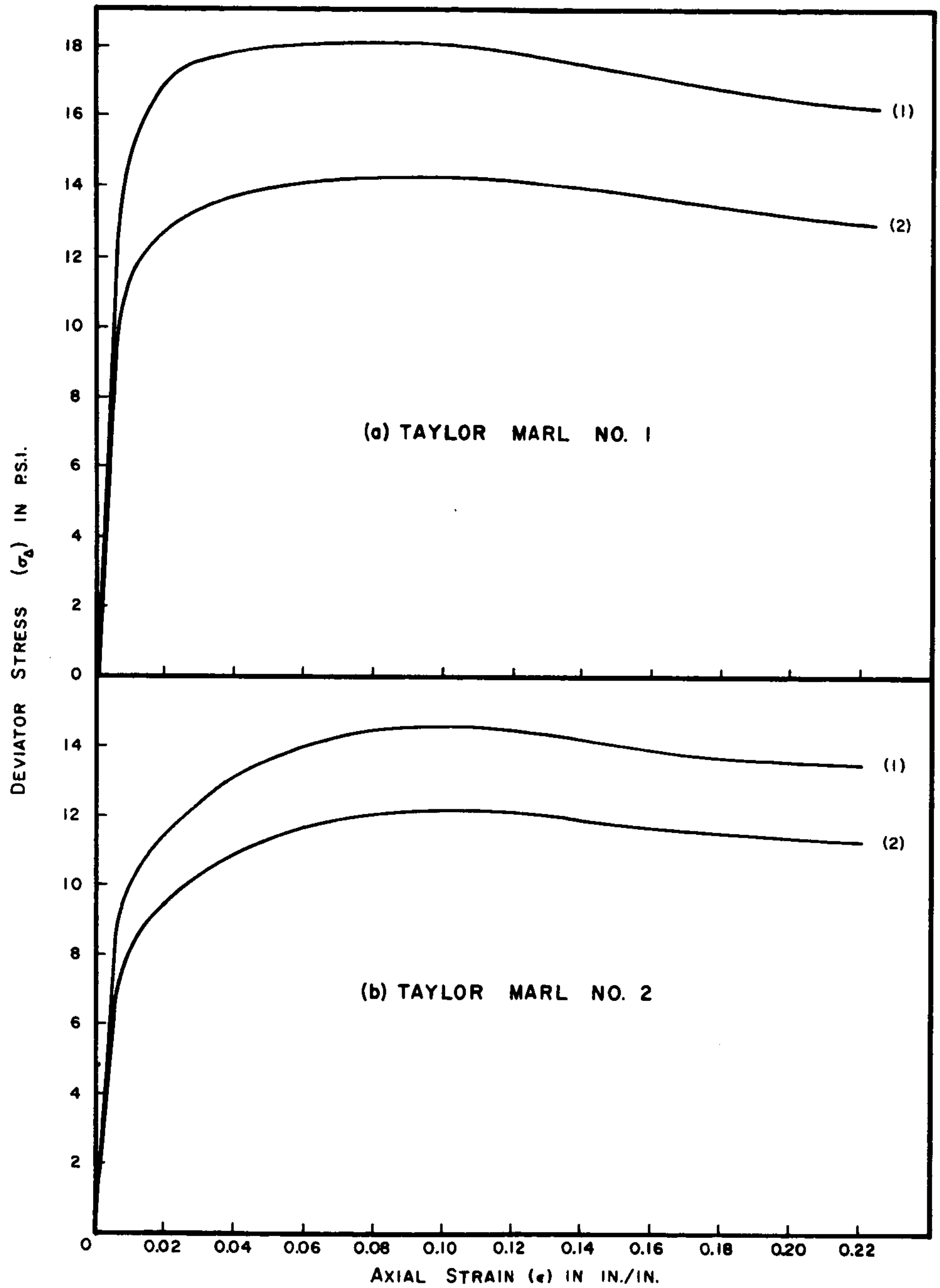
STRESS-STRAIN CURVES FOR OVER-CONSOLIDATED CLAYS

The stress-strain curves for over-consolidated samples of Taylor Marl No. 1 and 2 are shown in Fig 43. Each curve is the average of two identical tests. The results are averaged graphically. The maximum variation in moisture content between any two identical tests was in the order of 0.64%.

As the over-consolidation ratio increased, the initial linear portion of the curve and its slope decreased. Also the curve peaked to a lesser degree.

The effect of plasticity on the stress-strain curves of over-consolidation clays, with the same σ_o and O.C.R., is observed by comparing curves (a) and (b) in Fig 43. This effect has already been presented in Art. 7.3.

The deviator stresses were determined using areas computed on the assumption of no volume change and cylindrical shape during the test.



(1) O.C.R. = 2
(2) O.C.R. = 4

FIG. 43. STRESS-STRAIN CURVES FOR OVERCONSOLIDATED CLAYS

APPENDIX F

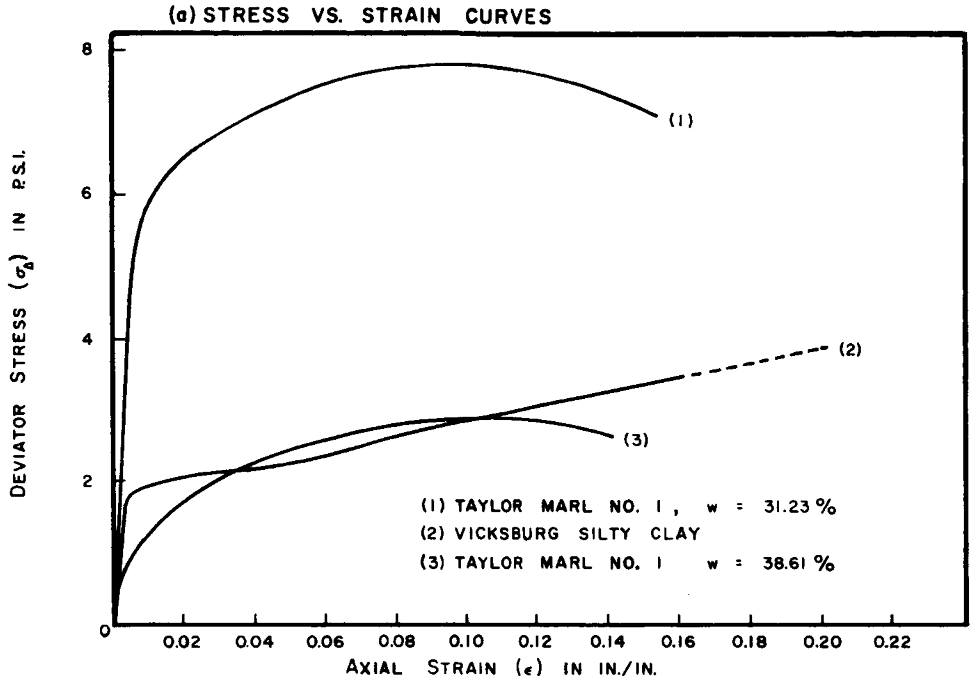
STRESS-STRAIN CURVES AND VOLUME CHANGES FOR UNCONFINED TESTS ON CLAYS

i. Stress-Strain Curves

The stress-strain curves for unconfined compression tests on clays are shown in Fig 44(a). Each one of these curves is an average of two identical tests. As mentioned before, the stresses are computed on the basis of areas determined from the measured lateral deformations. When compared with stresses computed using the conventional assumptions of no volume change and perfect cylindrical shape during the test, the difference was found to be very small at all strains experienced in these tests. This difference increased as strain got higher. The difference in the maximum stresses computed by both methods was much less than the small difference between the maximum stresses of the two identical tests. The maximum stresses in both methods occurred at the same axial strain. Therefore it is concluded that the assumptions made for computing stresses in unconfined and quick triaxial tests are justified, particularly in the low range of strains.

ii. Volume changes During the Tests

The assumption of no volume change, usually made in the conventional unconfined and triaxial tests on saturated clays, was investigated. It was found that small values of volume change do occur in the test specimen during the test. Fig 44(b) shows the results of unconfined tests on clays. The volume change in any test increased with the increase in strain. The



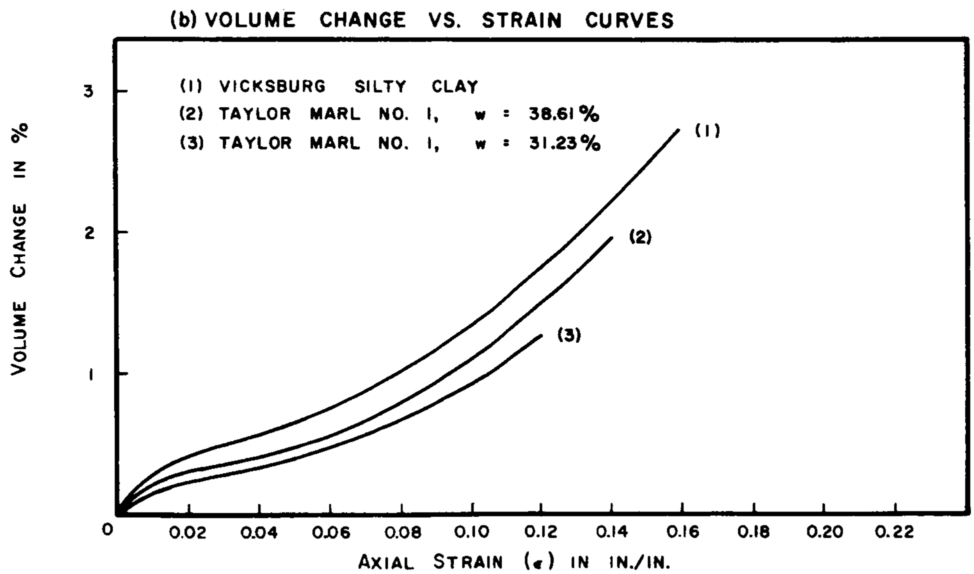


FIG. 44. UNCONFINED COMPRESSION TESTS ON CLAYS

volume change that took place at any strain was higher for the clay with the lower plasticity index and also for the clay with the higher moisture content. The value of the volume change at ten per cent axial strain was, in the average, equal to about one percent of the original volume of the specimen before the test started. At fourteen per cent strain, the average volume change became about 2 per cent. Therefore the validity of assuming zero volume change depends in the first place on the level of strain. The discrepancy between assumed and actual conditions are smaller at low strains. The author expects that the confining pressure may probably have an effect on volume changes. This and other variables should thoroughly be investigated before final conclusions are drawn.

BIBLIOGRAPHY

REFERENCES

1. Barden, L., "Stresses and Displacements in a Cross-Anisotropic Soil," Geotechnique, London, Vol. XIII, No. 3, September, 1963, pp. 198-210.

2. Bishop, A. W., and Henkel, D. J., "A Constant-Pressure Control for the Triaxial Compression Test," Geotechnique, London, Vol. III, No. 8, Dec., 1953, pp. 339-344.

3. Bishop, A. W., and Henkel, D. J., The Measurement of Soil Properties in the Triaxial Test, Second Edition, Edward Arnold Publishers LTD, London, 1962.

4. Borg, S. F., Fundamentals of Engineering Elasticity, D. Van Nostrand Company, New York, 1962, p. 46.

5. Borowicka, H., "The Mechanical Properties of Soils," Proceedings, 5th International Conference on Soil Mechanics and Foundation Engineering, Paris, Vol. I, Div. 1, No. 6, July, 1961, p. 39.

6. Broms, B. B., and Casbarian, A.O.P., "Effects of Rotation of the Principal Stress Axes and of the Intermediate Principal Stress on the Shear Strength of a Remolded Clay," Cornell University, 1964, Unpublished Paper.

7. Casagrande, A., and Wilson, S. D., "Effect of Rate of Loading on the Strength of Clays and Shales at Constant Water Content," Geotechnique, London, Vol. II, No. 3, June, 1951, pp. 251-263.

8. Conforth, D. H., "Some Experiments on the Influence of Strain Conditions on the Strength of Sand," Geotechnique, London, Vol. XIV, No. 2, June, 1964, p. 143.

9. Dawson, R. F., Laboratory Manual in Soil Mechanics, Second Edition, Pitman, New York, 1959.

10. De Jone, G. DeJosselin, and Gueze, E.C.W.A., "A Capacitive Cell Apparatus," Proceedings, 4th International Conference on Soil Mechanics and Foundation Engineering, London, Vol. I, Div. 1, No. 13, 1957, p. 52.

11. Dunlap, W. A., "A Report on a Mathematical Model Describing the Deformation Characteristics of Granular Materials," Texas Transportation Institute, Texas A & M University, Technical Report No. 1, Project 2-8-62-27, 1962.

12. Escaria, V., and Uriel, S., "Optical Methods of Measuring the Cross-Section of Samples in the Triaxial Test," Proceedings, 5th International Conference on Soil Mechanics and Foundation Engineering, Vol. I, Div. 1, No. 15, 1961, p. 89.

13. Folque, J., "Rheological Properties of Compacted Unsaturated Soils," Proceedings, 5th International Conference on Soil Mechanics and Foundation Engineering, Vol. I, Div. 1, No. 19, 1961, p. 113.

14. Grainer, G. D., and Lister, N. W., "A Laboratory Apparatus for Studying the Behavior of Soils Under Repeated Loading," Geotechnique, London, Vol. XII, No. 1, March, 1962, pp. 3-14.

15. Habib, P., "Influence of the Variation of the Average Principal Stress Upon the Shearing Strength of Soils," Proceedings, 3rd International Conference on Soil Mechanics and Foundation Engineering, Zurich, Vol. I, 1953, pp. 131-136.

16. Hardin, B. O., "Dynamic Versus Static Shear Modulus for Dry Sand," Materials Research and Standards, May, 1965.

17. Heierli, W., "Inelastic Wave Propagation in Soil Columns," Journal of the Soil Mechanics and Foundations Division, Proceedings, ASCE, Vol. 88, No. SM6, Part 1, Dec., 1962, p. 3347.

18. Henkel, D. J., "The Effect of Overconsolidation on the Behavior of Clays During Shear," Geotechnique, London, Vol. VI, No. 4, Dec., 1956, pp. 139-150.

19. Henkel, D. J., and Gilbert, G. D., "The Effect of the Rubber Membrane on the Measured Triaxial Compression Strength of Clay Samples," Geotechnique, London, Vol. III, No. 1, March, 1952, pp. 20-29.

20. Khuri, F. I., Elastic and Plastic Properties of Soils Influencing the Design of Rigid Pavements, Ph.D. Dissertation in C.E., Texas A & M University, College Station, June, 1954.

21. Krynine, D. P., Soil Mechanics, McGraw-Hill Book Company, New York, 1941.

22. Ladd, C. C., "Stress-Strain Modulus of Clay in Undrained Shear," Journal of the Soil Mechanics and Foundations Division, Proceedings, ASCE, Vol. 90, No. SM5, Part 1, September, 1964, pp. 103-132.

23. Lambe, T. W., Soil Testing for Engineers, John Wiley and Sons, Inc., New York, 1951.

24. Lo, K. Y., "Stress-Strain Relationship and Pore Water Pressure Characteristics of a Normally-Consolidated Clay," Proceedings, 5th International Conference on Soil Mechanics and Foundation Engineering, Vol. I, Div. 1, No. 37, 1961, p. 219.

25. Marsal, R. J., and Resines, J. S., "Pore Pressure and Volumetric Measurements in Triaxial Compression Tests," Proceedings, ASCE Research Conference on Shear Strength of Cohesive Soils, University of Colorado, Boulder, Colorado, June, 1960, p. 965.

26. Matlock, H., Fenske, C. W., and Dawson, R. F., "De-Aired, Extruded Soil Specimens for Research and for Evaluation of Test Procedures," American Society for Testing Materials, Bulletin No. 177, October, 1951.

27. Poulos, S. J., "Report on Control of Leakage in the Triaxial Test," Harvard Soil Mechanics Series, No. 71, 1964.

28. Reiner, M., Building Materials, Their Elasticity and Plasticity, Interscience Publishers, Inc., New York, 1954.

29. Roscie, K. H., Schofield, A. N., and Thurairajah, A., "Yielding of Clays in States Wetter than Critical," Geotechnique, London, Vol. XIII, No. 3, September, 1963, pp. 211-240.

30. Rowe, P. W., and Borden, L., "Importance of Free Ends in Triaxial Testing," Journal of the Soil Mechanics and Foundations Division, Proceedings, ASCE, Vol. 90, No. SM1, Part 1, January, 1964, pp. 1-27.

31. Sellards, E. H., Adkins, W. S., and Plummer, F. B., "The Geology of Texas," The University of Texas Bulletin No. 3232, Vol. 7, August, 1932.

32. Shockley, W. G., and Ahlvin, R. G., "Non-uniform Conditions in Triaxial Test Specimens," Proceedings, ASCE Research Conference on Shear Strength of Cohesive Soils, University of Colorado, Boulder, Colorado, June, 1960, p. 341.

33. Skempton, A. W., "The Bearing Capacity of Clays," Proceedings, Building Research Congress, London, Division I, Part III, 1951, pp. 180-189.

34. Sparrow, R. W., and Beaty, A.N.S., "An Instrumented Triaxial Base," Civil Engineering, July, 1964.

35. Spillers, W. R., and Stoll, R. D., "Lateral Response of Piles," Unpublished paper, prepared for Publication in the Proceedings of the ASCE, 1964.

36. Suklje, L., "The Equivalent Elastic Constants of Saturated Soils Exhibiting Anisotropic and Creep Effects," Geotechnique, London, Vol. XIII, No. 4, December, 1963, pp. 291-309.

37. Taylor, D. W., Fundamentals of Soil Mechanics, John Wiley and Sons, Inc., New York, 1948.

38. Taylor, D. W., "Shearing Characteristics of Clay," M.I.T. Report, 1951.

39. Terzaghi, K., "Principles of Soil Mechanics," Engineering News Record, Vol. 95, 1925.

40. Terzaghi, K., Theoretical Soil Mechanics, John Wiley and Sons, Inc., New York, 1943.

41. Waterways Experiment Station, "Coefficient of Earth Pressure At-rest and Poisson's Ratio," Vicksburg, Mississippi, 1955.

42. Waterways Experiment Station, "Poisson's Ratio Factor for Earth Pressure At-rest," Vicksburg, Mississippi, 1952.

43. Werner, R. R., A Study of Poisson's Ratio and the Elastic and Plastic Properties of Ottawa Sand, M.S. Thesis in C.E., Texas A & M University, May, 1957.

44. Whitman, R. V., and Richardson, A. M., "Time Lags in Pore Pressure Measurements," Proceedings, 5th International Conference on Soil Mechanics and Foundation Engineering, Vol. I, Div. 1, No. 69, 1961, p. 407.

45. Wilson, S. D., and Dietrich, R. J., "Effect of Consolidation Pressure on Elastic and Strength Properties of Clay," Proceedings, ASCE Research Conference on Shear Strength of Cohesive Soils, University of Colorado, Boulder, Colorado, June, 1960, pp. 419.

46. Wilson, S. D., and Sibley, E. A., "Ground Displacements Resulting from Air-Blast Loading," Preprint paper to be presented at the National ASCE Convention, Houston, Texas, February, 1962.

47. Wolfskill, L. A., and Buchanan, S. J., "Dynamic Stress-Strain Characteristics of Granular Materials," A Paper Delivered at the National Conference of ASCE, Houston, Texas, February, 1962.

48. Zbigniew, D., and Zebski, J., Nature and Properties of Engineering Materials, John Wiley and Sons, Inc., New York, 1959.

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